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FERTILIZATION OF LAKES BY AGRICULTURAL AND URBAN DRAINAGE

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THE lakes at Madison, Wis., are notorious for several reasons: first, because Henry Wadsworth Longfellow once wrote a complimentary poem about them; second, because of the highly undesirable algal blooms which of recent years have infested, in particular, the lakes in the chain below the city and which have driven lake-shore cottagers away and resulted in a general depreciation of lake-shore property values; third, because of the large-scale use of copper sulfate in an attempt to control the algal blooms in Lakes Monona, Waubesa and Kegonsa, having a combined area of 13.5 square miles.

HISTORICAL ASPECTS

Reference to Figure 1, which is a map of the Madison lakes area, will aid materially in correlating the following events. The city of Madison, in the main, is located on the narrow strip of land between Lakes Mendota and Monona. Prior to 1884 the city had no sewerage system and only certain residences adjacent to the lake shores disposed of their wastes by dilution in the nearby waters. In 1884 the city started construction of a sanitary sewer system. Drainage of raw sewage into Lakes Mendota and Monona was practised until 1899, when the first sewage-treatment plant using chemical processes was placed in operation. At that time all discharges into Lake Mendota were discontinued and effluent from the sewage-treatment plant was run into the Yahara River and thence into Lake Monona. From 1902 to 1926 all sewage was treated by some form of biological treatment and the effluent was added to Lake Monona.

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Note: This paper was read at a Symposium on Fertilization of Aquatic Areas, held in Boston, December 30, 1946, by the Limnological Society of America. The author used this paper as the basis for a talk before the Chemists' Round Table of the New England Water Works Association on November 14, 1946.

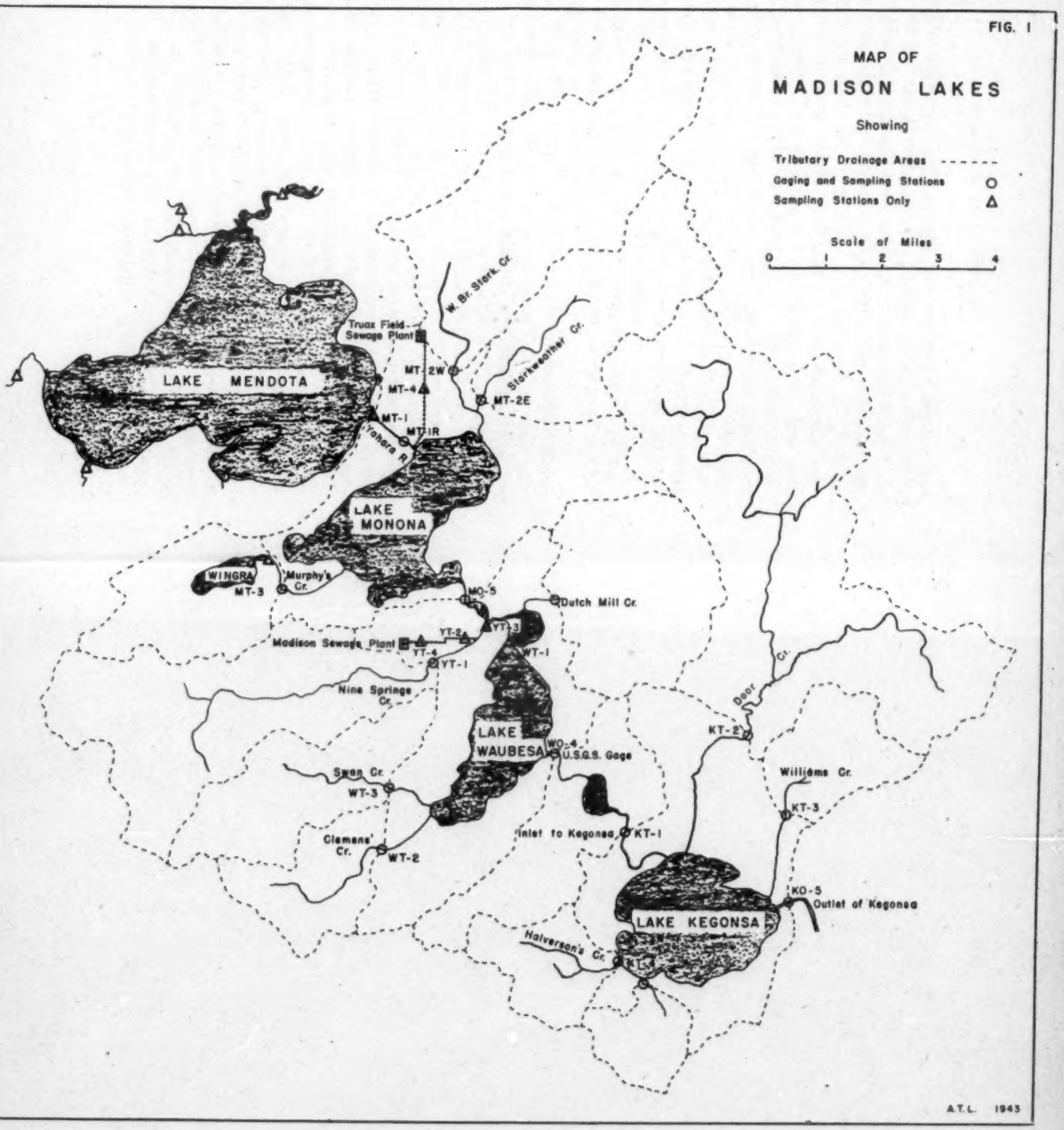
By 1918 conditions of algal growth and resultant nuisances had become so bad in Lake Monona that the Mayor and City Council of Madison requested the engineering firm of Alvord and Burdick of Chicago to investigate the situation (1). After the engineers had made an extensive study of the problem they reported: 1.—"All the evidence appears to show conclusively that the offensive smells from Lake Monona during the summer of 1918 were caused by algae, produced in great abundance in lake waters"; 2.—"We believe the indirect influence of the sewage effluent (pop. 35,000) in stimulating algal growths to be proportionally unimportant"; and 3.—The engineers indicated the sewage-plant effluent was only 1/20 as important as agricultural drainage in the odor problem. Furthermore, these conclusions were confirmed by two well-known scientists of the University of Wisconsin in an appendix to the report.

Copper-sulfate treatment of Lake Monona for control of algal blooms and the curtailment of nuisance conditions was started in 1918 and has been used each year since.

During 1925-26 the city of Madison built an Imhoff-trickling-filter sewage-treatment plant, about 5 miles away from the main part of the city, on land between Lakes Monona and Waubesa and the plant was placed in operation in 1926, with effluent discharging into the Yahara River and thence into Lake Waubesa. In 1935 activated-sludge treatment and separate sludge-digestion facilities were added to the new plant and in 1936 the old plant with drainage to Lake Monona was abandoned and all sewage was pumped to the new plant for treatment. Shortly thereafter the lake-shore residents of Lakes Waubesa and Kegonsa complained of odor nuisances and copper-sulfate treatment of those lakes was started.

Because of unsatisfactory control of the nuisance problem in Lakes Waubesa and Kegonsa, and because of the feeling on the part of many people that the nuisance problem was caused by the sewage-plant effluent and that the city of Madison had acted in an unethical manner in moving its sewage-treatment facilities from the drainage area tributary to Lake Monona downstream to the drainage area tributary to Lake Waubesa, the lake-shore residents of the lower lakes, through the medium of their Lake Improvement Associations, introduced a bill into the Wisconsin legislature during the 1941 ses-

FIG. 1


MAP OF
MADISON LAKES
Showing
Tributary Drainage Areas
Gaging and Sampling Stations
Sampling Stations Only
Scale of Miles
0 1 2 3 4
LAKE MENDOTA
LAKE MONONA
LAKE WAUBESA
LAKE KEGONSA
Truax Field Sewage Plant
W. Br. Starkweather Cr.
Starkweather Cr.
MT-2W
MT-4
MT-2E
MT-1
MT-1R
Yahara R.
WINGRA
MT-3
Murphy's Cr.
MO-5
Dutch Mill Cr.
Madison Sewage Plant
YT-2
YT-3
YT-4
YT-1
WT-1
Nine Springs Cr.
Door Cr.
KT-2
WO-4
U.S.G.S. Gage
Williams Cr.
Spring Cr.
WT-3
KT-3
Inlet to Kegonsa
KT-1
Clemons' Cr.
WT-2
KO-5
Outlet of Kegonsa
Halverson's Cr.
KT-4
A.T.L. 1943

sion, which was designed to make it unlawful for the Madison Metropolitan Sewerage District to dispose of sewage or treated sewage in Lakes Monona, Waubesa or Kegonsa. The bill was passed by the legislature and was vetoed by the Governor, after an understanding had been reached that the Madison Sewerage District would finance an investigation conducted by a committee appointed by the Governor.

The Governor's Committee appointed to investigate the odor-nuisance problem in the Madison lakes early concluded that the intense algal growths occurring in those lakes were related to the fertilizing matters reaching the lakes. Since the relative amounts contributed by domestic wastes and by agricultural drainage was a much-debated matter, based on very scanty data, it was decided to obtain data to settle this point. Consequently, all important streams and discharges tributary to Lakes Monona, Waubesa and Kegonsa were placed under survey.

During 1942, the sewage-treatment plant abandoned by the city in 1936 was rejuvenated by the U. S. Army and placed in operation during October, 1942, to treat the wastes of Truax Field, an A.A.F. training school. The effluent was discharged into Lake Monona.

At the beginning of the survey, analyses were run for nitrogen and phosphorus in organic and inorganic forms, as well as for potassium. It was soon found that potassium was present in appreciable quantities in all the lake waters and showed no marked variations seasonally. Consequently it was dropped from consideration.

Although the survey was continued for a period of two years, the data for the second year were somewhat more complete and, since they substantiated very well the data obtained during the first year, this paper will deal mainly with the data obtained during the second year of the survey.

The results presented in this paper have been reported in detail (2), (3), (4), (5).

Results of Fertilization Survey

A summary of the results obtained for nitrogen and phosphorus in the fertilization survey during the second year is shown in Figures 2 and 3.

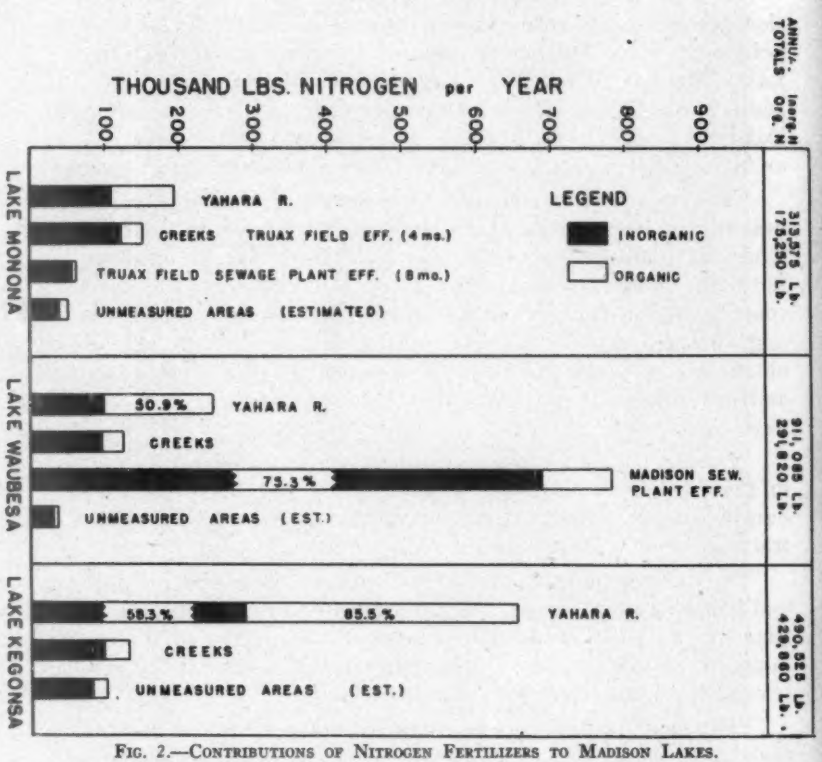


FIG. 2.—CONTRIBUTIONS OF NITROGEN FERTILIZERS TO MADISON LAKES.

Contributions of Nitrogen

The data in Figure 2 show that Lake Monona received a total of 488,825 lb of nitrogen during the year. Drainage from Lake Mendota via the Yahara River and from creeks constituted the major contributions of inorganic nitrogen (NH_3–N, NO_2–N and NO_3–N) and of organic nitrogen. The exact percentage is difficult of evaluation, because Truax Field sewage-plant effluent was diverted to one of the creeks during 4 months of the year and because the contribution shown for the Yahara River included some increase over that released from Lake Mendota, by reason of contamination by

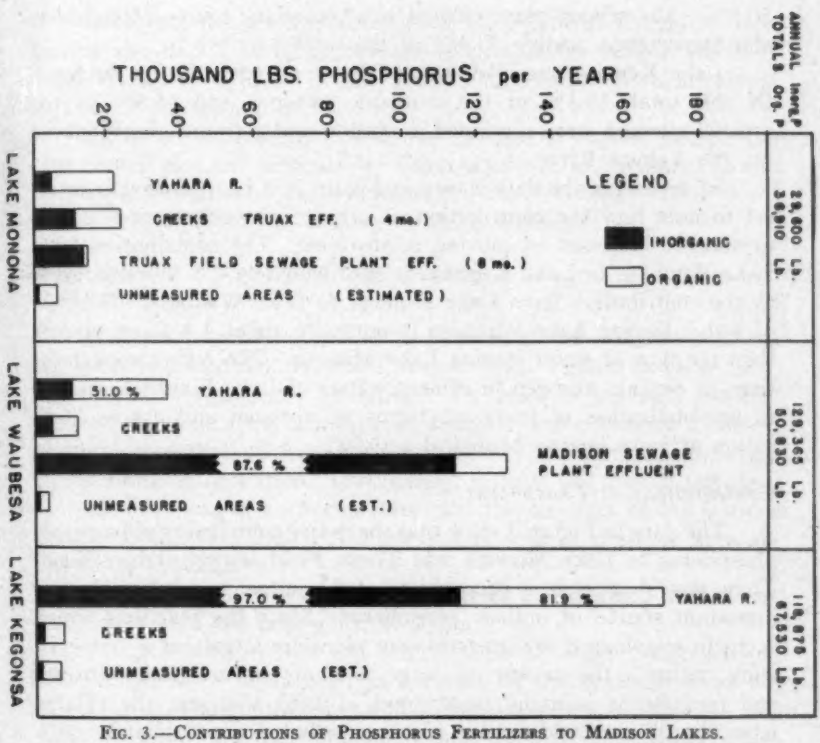


FIG. 3.—CONTRIBUTIONS OF PHOSPHORUS FERTILIZERS TO MADISON LAKES.

industrial wastes while flowing through the city. Allowances for these variations, based on the best data available, indicated that approximately 56% of the inorganic nitrogen and 88% of the organic nitrogen came from sources other than urban drainage.

For Lake Waubesa, which received 1,202,905 lb of nitrogen, the data show the biologically treated sewage of the city of Madison to be the chief contributor of inorganic nitrogen—namely, 75.3%. This value is known to be low, as one of the creeks is known to be contaminated by drainage from sludge-disposal practices at the sewage-treatment plant. The waters leaving Lake Monona via the Yahara River are the chief contributors of organic nitrogen, namely

50.9%. The sewage-plant effluent is a secondary source of considerable importance, adding 32.4% of the total.

Lake Kegonsa received 920,385 lb of nitrogen during the year. Of this total, 58.3% of the inorganic nitrogen and 85.5% of the organic nitrogen were contained in outflow water from Lake Waubesa via the Yahara River.

In reviewing the data shown in Figure 2, it is of particular interest to note how the contribution of organic nitrogen from one lake to another increases on moving downstream. The contribution from Lake Waubesa to Lake Kegonsa is approximately 2.5 times as great as the contribution from Lake Monona to Lake Waubesa. The flow of water leaving Lake Waubesa is normally about 1.4 times greater than the flow of water leaving Lake Monona. The higher concentrations of organic nitrogen in effluent waters of Lake Waubesa are due to immobilization of inorganic forms of nitrogen and are an indication of more intense biological activity.

Contributions of Phosphorus

The data in Figure 3 show that the major contributor of inorganic phosphorus to Lake Monona was Truax Field sewage-plant effluent. They also show that a properly treated sewage is a relatively unimportant source of organic phosphorus. Since the reactions which occur in a biological sewage-treatment plant are largely of a "mineralizing nature", the release of large amounts of inorganic nitrogen and phosphorus is easily understood. During the year the effluent from the Truax Field sewage plant contributed approximately 68% of the inorganic phosphorus and about 4% of the organic phosphorus. The remainder of the organic phosphorus was furnished in almost equal quantities by drainage from Lake Mendota and from creeks draining agricultural areas.

In the case of Lake Waubesa, the importance of urban drainage as a source of inorganic phosphorus is again demonstrated. It will be seen that 87.6% originated in the effluent of the Madison sewage-treatment plant. The principal contributor of organic phosphorus was the outflow water of Lake Monona, which furnished 51.0%. The effluent of the sewage plant was the secondary contributor of greatest importance.

Lake Kegonsa was found to receive 97.0% of its inorganic phosphorus and 81.9% of its organic phosphorus in the outflow water of Lake Waubesa. This, of course, reflects the condition which existed in Lake Waubesa and its causes.

Again it is of interest to note the increase of organic phosphorus contributed to Lake Kegonsa by Lake Waubesa, as compared to the amount contributed by Lake Monona to Lake Waubesa, further proof of the greater biological activity in Lake Waubesa.

Contributions of Fertilizing Matters from Agricultural Areas

It was impossible, of course, to measure all the drainage reaching the lakes from agricultural areas or even from a given drainage area, because of topographical limitations. In order to evaluate properly the contributions from unmeasured areas, it was necessary to determine the contributions from measured areas which involved no domestic or industrial pollution. Fortunately, at least one stream tributary to each lake was of such character and the amounts of the various fertilizing elements contributed per square mile of drainage area are shown in Table 1.

The values given for unmeasured areas in Figures 2 and 3 were calculated from the data given in Table 1.

It will be noted that the contributions of organic nitrogen per square mile were fairly uniform for each of the drainage areas but those for inorganic nitrogen and phosphorus varied considerably. The highest values for the inorganic forms were from localities which had large areas of marsh lands developed for agricultural purposes. Such areas release abnormal amounts of nitrate nitrogen under proper conditions and appear to be less retentive of phosphate fertilizers than ordinary farm lands.

Contributions of Fertilizing Matters from Biologically Treated Domestic Sewage

Analysis of the data obtained during the survey is difficult, because the sewage of Madison was treated in conjunction with chemically treated slaughter-house wastes. The best information available indicated that the contributing population was about 90,000

and that the packing industry supplied about 30% of the nitrogen and 10% of the phosphorus. Thus, with these allowances, it has been estimated that biologically treated domestic sewage would contain approximately 6.0 lb of nitrogen and 1.2 lb of phosphorus per capita per year. On this basis, it requires approximately 750 persons to supply an amount of nitrogen equivalent to that released by one square mile of agricultural lands in the Madison area and only 212 persons to supply the phosphorus equivalent. These comparisons are shown in Table 2.

TABLE 1.—CONTRIBUTIONS OF FERTILIZING ELEMENTS IN DRAINAGE FROM AGRICULTURAL LANDS

Drainage area tributary to	Pounds contributed per year per square mile			
	Inorganic nitrogen	Organic nitrogen	Inorganic phosphorus	Organic phosphorus
Lake Monona	2,800	1,035	35	
Lake Waubesa	3,130	1,180	62	186
Lake Kegonsa	4,100	1,150	62	200

TABLE 2.—COMPARISON OF FERTILIZING ELEMENTS SUPPLIED BY AGRICULTURAL DRAINAGE AND BY BIOLOGICALLY TREATED SEWAGE TRIBUTARY TO THE MADISON LAKES

Fertilizing element	Pounds per sq mi per year	Pounds per capita per year	Population equivalent per sq mi
Nitrogen (total)	4,500	6.0	750
Phosphorus (total)	255	1.2	212

Relative Fertilization of Madison Lakes

Although the above data serve to illustrate the relative significance of urban and agricultural fertilization of lakes, they do not serve the purpose of showing to what degree the various lake waters are fertilized. Tables 3 and 4, presented for this purpose, show that total nitrogen additions range from about 130 lb per acre per year in Lake Monona to 590 lb per acre per year in Lake Waubesa. The total phosphorus additions range from 19 lb per acre per year in Lake Monona to 88 lb per acre per year in Lake Waubesa. These quantities, especially those for Lake Waubesa, are greatly in excess

of normal applications to farm lands, or even to lakes and ponds where fish are cultivated, and indicate why the Madison lakes are such active producers of algal blooms.

The effect of these high rates of fertilization is shown in Table 5, which gives minimum and maximum values for inorganic nitrogen and phosphorus in the effluent waters of the Madison lakes during the course of a year. It will be noted that the inorganic nitrogen in the surface waters reaches approximately the same minimum value sometime during the course of the year, but that the maximum values reached vary considerably and the lakes receiving the greatest amounts of domestic drainage develop the highest concentrations. On the other hand, Lakes Waubesa and Kegonsa showed abundant amounts of inorganic phosphorus at all times, while the other lakes all developed summer minima of less than 0.01 ppm and slight increases during the winter months, depending upon the degree of

TABLE 3.—FERTILIZATION OF MADISON LAKES—NITROGEN BASIS

Lake	Year	Pounds of inorganic nitrogen		Pounds of organic nitrogen		Pounds of total nitrogen per acre
		Per lake	Per acre	Per lake	Per acre	
Monona	1942-43	289,030	83	153,220	44	127
Monona	1943-44	313,570	90	175,250	50	140
Waubesa	1942-43	928,225	457	265,300	131	588
Waubesa	1943-44	911,085	448	291,818	143	591
Kegonsa	1942-43	605,415	192	392,975	125	317
Kegonsa	1943-44	490,525	156	429,860	137	293

TABLE 4.—FERTILIZATION OF MADISON LAKES—PHOSPHORUS BASIS

Lake	Year	Pounds of inorganic phosphorus		Pounds of organic phosphorus		Pounds of total phosphorus per acre
		Per lake	Per acre	Per lake	Per acre	
Monona	1942-43	23,510	6.7	——	—	—
Monona	1943-44	29,600	8.5	36,910	10.5	19.0
Waubesa	1942-43	126,610	62.4	——	—	—
Waubesa	1943-44	129,365	63.6	50,830	25.0	88.6
Kegonsa	1942-43	109,045	34.8	——	—	—
Kegonsa	1943-44	118,875	37.7	67,530	21.5	59.2

sewage fertilization. Lakes Waubesa and Kegonsa usually showed their lowest concentration of inorganic phosphorus during the early spring and their highest concentration during August and September —quite the reverse of conditions with respect to inorganic nitrogen.

During the second year of the survey, in order to obtain a better picture of the condition which existed in the Madison lakes, surface or effluent samples were collected at intervals of about 5 weeks from 12 other lakes in southeastern Wisconsin. The samples were subjected to chemical analysis and the results are also shown in Table 5. These data show that only Lakes Koshkonong, Delevan and Nagawicka, together with upper Nemahbin, had concentrations of inorganic nitrogen approaching those shown by Lakes Waubesa and Kegonsa. These same lakes show appreciable amounts of inorganic phosphorus during the winter months. All are known to receive urban drainage and all produce nuisance blooms of algae annually, except Lake Koshkonong, which has had a relatively good record. Lake Koshkonong is a bit unusual, in that it is very shallow and, because of the size of the Rock River, may be considered simply a widening in the river. Of the other lakes, all except Lake Pewaukee are normally of good behavior. Its behavior remains somewhat of an enigma among the lakes studied. One plausible explanation may be that the lake is being fed from the bottom to a great extent, due to raising the water level approximately six feet and flooding large marsh areas, which represent nearly 50% of the total lake area at the present time.

Accumulative Fertilization of Lakes

It is quite generally understood that a lake, through the biological life it harbors, converts inorganic fertilizing elements into organic forms as a part of living organisms, which in the main have a density greater than that of water and on death settle to the bottom of the lake, except for that part which escapes in the effluent. The organic matter so deposited is subjected, of course, to attack by saprophytic organisms, which decompose the organic matter, releasing much of the fertilizing matter to the waters above in inorganic forms once more. The deposits of previous years continue to release fertilizing matters also and the accumulative effect of the total bottom deposit is to release nearly as much fertilizing matter each year as is

added to the bottom. Thus, if we are able to measure the amounts of fertilizing matters retained by a lake we have an indirect measure of the approximate amount released from bottom deposits.

A study of the retention of nitrogen in the Madison lakes is shown in Table 6. These data show considerable variation in the amount and in the percentage retained during the two years of the survey. This was probably due to the fact that copper-sulfate treatment was used to control algal blooming and such treatment undoubtedly effects, in general, the immobilization of inorganic forms of nitrogen by arresting or slowing down biological action. The lowest percentage noted, 30.4, however, is far in excess of the value reported by Mortimer for Lake Windermere (6).

The fact that Lake Waubesa, the smallest lake of the three included in Table 6, retained the greatest amount of nitrogen is fur-

TABLE 5.—COMPARISON OF EFFLUENT WATERS FROM 17 SOUTHEASTERN WISCONSIN LAKES

(based on at least 10 samplings over a 12-month period)
[Results in ppm]

Lake	Inorganic nitrogen			Inorganic phosphorus		
	Min.	Max.	Av.	Min.	Max.	Av.
Mendota	0.05	0.31	0.17	<0.01	0.025	0.018
*Monona	0.07	0.50	0.33	<0.01	0.07	0.041
*Waubesa	0.07	1.24	0.79	0.27	0.54	0.38
*Kegonsa	0.08	0.93	0.35	0.26	0.49	0.33
Wingra	0.07	0.53	0.26	<0.01	0.02	0.012
Koshkonong	0.12	1.32	0.39	<0.01	0.06	0.019
*Delevan	0.06	0.87	0.31	<0.01	0.07	0.023
Geneva	0.05	0.15	0.10	<0.01	<0.01	<0.01
Como	0.09	0.22	0.14	<0.01	0.01	<0.01
Lauderdale (Mill)	0.06	0.18	0.13	<0.01	<0.01	<0.01
*Pewaukee	0.06	0.22	0.15	<0.01	?	<0.01
*Nagawicka	0.08	0.88	0.37	<0.01	0.035	0.016
*Upper Nemahbin	0.05	0.65	0.24	<0.01	0.02	0.013
Okauchee	0.05	0.19	0.13	<0.01	0.01	<0.01
Oconomowoc	0.04	0.15	0.12	<0.01	0.01	<0.01
Lac La Belle	0.07	0.30	0.19	<0.01	0.01	<0.01
Rock	0.07	0.15	0.10	<0.01	0.01	<0.01

*Indicates lakes which produce nuisance blooms regularly.

TABLE 6.—RETENTION OF NITROGEN IN MADISON LAKES

Lake	Year	Total Nitrogen In (lb)	Retained (lb)	% retained
Monona	1942-43	394,390	119,800	30.4
Monona	1943-44	488,820	244,590	50.2
Waubesa	1942-43	1,101,015	667,315	60.5
Waubesa	1943-44	1,202,900	540,680	45.1
Kegonsa	1942-43	897,490	277,340	30.9
Kegonsa	1943-44	920,385	355,515	38.6

ther evidence that it is the most active biologically of the Madison lakes.

The role of bottom deposits in supplying nutrient elements and the influence of sewage thereon is best shown by comparing the vertical distribution of inorganic nitrogen and phosphorus in Lakes Mendota and Monona during the winter and summer stagnation periods. Such data, presented in Figures 4 and 5, show that the deeper waters of Lake Monona become much more heavily charged with nutrients than do the waters of Lake Mendota, which receives very little urban drainage and normally is much less productive, biologically.

BIOLOGICAL RESPONSE TO FERTILIZATION

Biological studies were conducted by Dr. James B. Lackey, at that time Senior Biologist of the Cincinnati Experiment Station of the U. S. Public Health Service, during the last 20 months of the survey. His conclusions were that the most heavily fertilized lakes were the most productive of blooms, especially nuisance blooms of the blue-green variety. For example, Lake Waubesa, the most heavily fertilized of the lakes under survey, had 112 blooms during the last year, 48 being due to blue-green algae and 5 of the 48 being due to *Microcystis aeruginosa*, which is especially obnoxious. In contrast, Lake Mendota had a total of 22 blooms during the same period and only two of them were due to blue-green algae (3).

Furthermore, Lakes Kegonsa and Waubesa are known to develop intense blooms of green algae under ice cover, when snow cover is slight or absent and light conditions are favorable. Such blooms have resulted in severe fish kills on occasion.

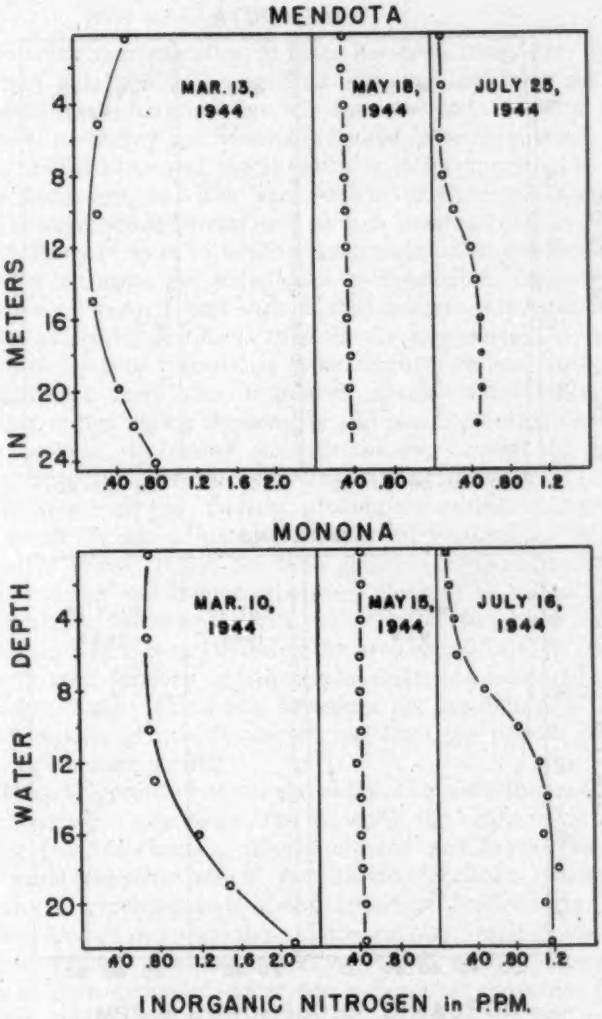


FIG. 4.—VERTICAL DISTRIBUTION OF NUTRIENT ELEMENTS IN DEEP LAKES—INORGANIC NITROGEN.

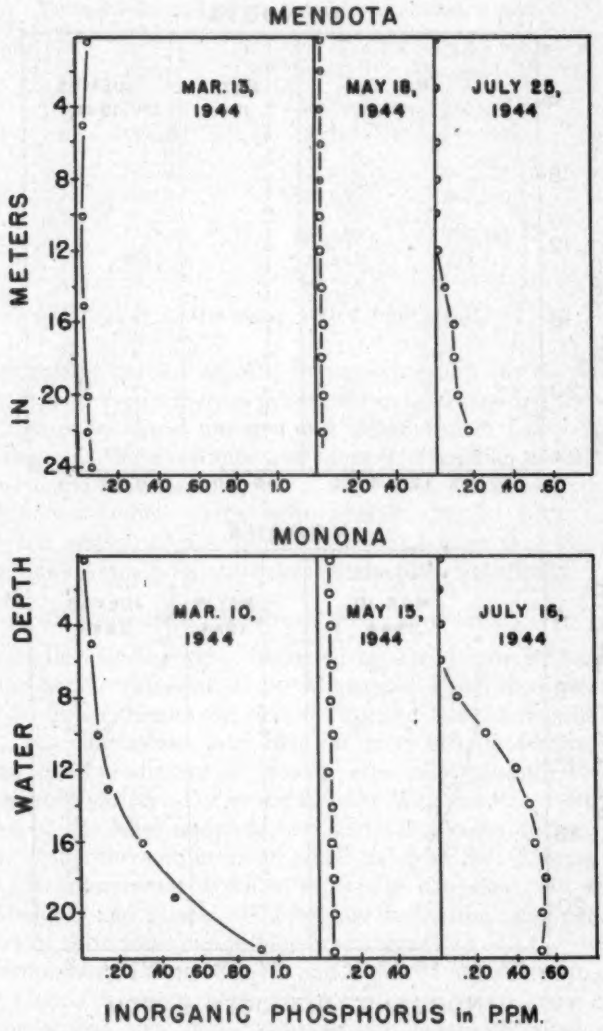


FIG. 5.—VERTICAL DISTRIBUTION OF NUTRIENT ELEMENTS IN DEEP LAKES—INORGANIC PHOSPHORUS.

Discussion

Domestic eutrophication of lakes has been recognized for many years and has been well correlated with the fertilizing matters in, and pollutional characteristics of, domestic and industrial wastes. Somehow the feeling has become prevalent in some quarters that all that is needed to correct the situation is to incorporate a system of sewage treatment, to clean such waters of their pollutional load. Unfortunately, sewage treatment, as it is practised today, is largely a mineralizing process, in which organic substances are broken down to release nitrogen and phosphorus as ammonium compounds and phosphate compounds, and such treated sewages contain much more "readily available" fertilizers than do the raw wastes. As a result, the growth response created is often much more spontaneous.

Eutrophic deep lakes, in general, develop anaerobic conditions in deeper waters during the summer and possibly during winter stagnation periods. Untreated domestic wastes, because of the great amount of organic matter they contain, cause anaerobic conditions to develop sooner and, thereby, prolong the period of anaerobiosis. As a result, increased amounts of nutrient matters are released to the waters above. It has not been generally recognized that highly treated sewages can produce the same effect in an indirect manner, by stimulating excessive growths which, upon death and subsequent decay, create similar conditions in the deep waters of such lakes. The normally good behavior of oligotrophic lakes, as compared to eutrophic lakes, makes it desirable to reduce the possibilities of domestic eutrophication, or at least shorten and limit the periods of anaerobiosis as much as possible.

Thus, the problem of sewage disposal in some instances appears to be entering a new phase, one in which the center of attention is shifting from biochemical oxygen demand and oxygen balances in their usual sense to one of fertilization, increased biological productivity of receiving waters, and attendant problems. In the case of lakes, as well as rivers, this stimulation may result in the development of serious nuisance conditions, which greatly depreciate the value of such bodies of water for recreational purposes, for water supplies, and as desirable places around which to live during the summer months.

It is too much to expect that the lakes of this country can be restored to their pristine glory in the present state of our agricultural and urban development. However, some measure of control seems possible, through reduction of the quantity of fertilizing matters reaching critical bodies of water from domestic and agricultural sources. For domestic wastes, such control may take form in tertiary treatment of sewage to remove the nutrient elements or, in cases where industrial wastes of carbonaceous character are available, the domestic and industrial wastes may be so balanced for treatment that recovery of the nutrient elements can be accomplished in two stages of treatment, so that effluents relatively free from fertilizing elements can be produced.

To reduce the contribution from agricultural areas, improved farming methods should be encouraged. Reduction of surface run-off and soil erosion by contour-farming methods and education of farmers as to proper times to apply animal manures, so as to avoid losses during frozen-ground conditions, should be fostered.

The author feels that he would be remiss if he did not point out some of the conclusions he has reached through his experiences on the Madison survey. These are offered, not because he feels he is an especially well-qualified limnologist, but rather to add the viewpoint of an outsider.

In the first place, a "round-the-calendar" record is of vital importance in fertilization surveys. So often attempts at explaining biological happenings are based on "post-mortem" evidence or data obtained on surface samples collected in the summer-time. The situation is quite analogous to the one which exists in epidemiological studies of outbreaks of food poisoning. Most often the incriminating material has been consumed and it is only through analysis of stomach contents or excreta of some of the victims that a reasonable solution may be deduced. On the other hand, after a biological eruption has occurred, it is often as futile to seek for the answer in the surface waters of a lake as to analyze the contents of a room in which a contaminated food has been eaten and departed in the stomachs of the participants.

A record of changes which occur in the deeper waters of a lake, especially in the hypolimnion of deep lakes, is necessary, if a complete

picture of sources of fertilizing matters is to be obtained. In this connection, it may be well to re-emphasize that a descending thermocline during the summer months gradually feeds nutrient, rich waters from the hypolimnion into the epilimnion, thus constantly enriching the surface waters. The importance of deep waters, especially those with anaerobic strata, as a source of carbon dioxide should not be overlooked.

The importance of ammonia nitrogen as a source of food material for phyto-plankton seems to have been overshadowed by the conception that nitrate nitrogen is *the* important form of nitrogen as fertilizer. The author cannot agree that nitrate nitrogen deserves all this credit. In fact, he feels that ammonia nitrogen is much the most important as a stimulant to explosive algal growths and may be a factor in determining the type of bloom produced.

Although *Microcystis aeruginosa* is the member of the blue-green algae which causes the greatest amount of trouble in the lakes at Madison, which receive urban drainage, it cannot be stated definitely that urban drainage is conducive to the development of this particular alga, as all lakes involved are treated with copper sulfate and the blooms of this particular blue-green alga which occur may be a result of its greater resistance to the toxic properties of the copper salt.

All the evidence obtained in the Madison survey lends support to the belief that phosphorus is a key element in determining the biological activity in a body of water. The data regarding critical levels of organic phosphorus are not too well correlated but, in the main, show that most of the lakes producing nuisance blooms have average concentrations in excess of 0.10 ppm. The data concerning critical levels of inorganic phosphorus, however, are much more decisive and indicate that nuisance conditions can be expected when the concentration of inorganic phosphorus exceeds or equals 0.01 ppm. Lakes Waubesa and Kegonsa, of course, never lack for phosphorus at any season of the year.

The importance of nitrogen in supporting algal blooms cannot be denied. A critical level of 0.30 ppm of inorganic forms is indicated from the data obtained. However, it should be pointed out that extensive algal growths were produced under laboratory conditions with plentiful supplies of phosphorus and deficient supplies of

nitrogen. Undoubtedly, nitrogen fixation, either bacterial or algal, bridges the deficiency and produces the nitrogen necessary for the synthesis of algal protein. In the absence of plentiful supplies of phosphorus, however, nitrogen fixation was found to be unimportant.

Another factor of importance in problems involving urban drainage is that of concentration. It should be appreciated that the concentration of nutrient elements which exists in biologically treated sewage (inorganic nitrogen 15-25 ppm, inorganic phosphorus 2-4 ppm) is 10-100 times greater than that of normal agricultural drainage. When such wastes enter a lake, they cause outflow of an equal volume (less evaporation) of relatively good water. Such a change can only result in an appreciable increase of the concentration of nutrient elements in the receiving water. On the other hand, the composition of normal agricultural drainage is more comparable to lake waters in the concentration of nutrient elements carried. Consequently, when it enters a lake, it displaces water with more nearly an equal nutrient content and the net gain to the lake is small. This factor is of tremendous importance in Lake Waubesa, where the sewage flow approximates 15% of the total annual inflow to the lake and often exceeds 50% of the flow during the summer months.

SUMMARY

1. Biologically treated sewage is an important source of inorganic forms of nitrogen and phosphorus fertilizers. For Lake Waubesa, whose inflowing waters consist of approximately 15% biologically treated sewage, 75.3% of its inorganic nitrogen and 87.6% of its inorganic phosphorus were contributed by effluent from the Madison, Wis., sewage-treatment plant.

2. In the Madison chain of lakes, organic forms of nitrogen and phosphorus fertilizers are contributed largely by outflow waters of the lakes, the amount released by each lake being a function of the degree of fertilization of the lake.

3. Agricultural drainage in the Madison area was found to contribute approximately 4,500 lb of nitrogen and 255 lb of phosphorus per square mile of drainage area per year. Biologically treated sewage was found to supply approximately 6.0 lb of nitrogen and 1.2 lb of phosphorus per capita per year. Thus, the biologically treated wastes

of 750 persons are equivalent to the agricultural drainage from one square mile of area, on a nitrogen basis, and similarly treated wastes of 212 persons are equivalent to drainage from the same area, on a phosphorus basis.

4. The lakes at Madison downstream from the city were found to be fertilized by nitrogen in amounts ranging from 127 to 588 lb per acre per year and by phosphorus in amounts ranging from 19.0 to 88.6 lb per acre per year.

5. Lakes Waubesa and Kegonsa were found to have excessive inorganic phosphorus at all times of the year, usually 0.25 ppm or more. All other lakes studied—15 of them—showed summer minima of less than 0.01 ppm.

6. All lakes receiving appreciable urban drainage showed winter maxima of inorganic phosphorus in excess of 0.02 ppm.

7. The Madison lakes were found to retain 30.4 to 60.5% of the nitrogen they received.

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*Copies of these reports may be obtained by interested parties by addressing requests to: Madison Metropolitan Sewerage Commission, 110 East Main St., Madison, Wis.

EXPERIENCE WITH *ANABAENA* IN KENOZA LAKE, HAVERHILL, MASSACHUSETTS

BY E. SHERMAN CHASE*

[*Given at Chemists' Round Table, December 19, 1946.*]

DURING the fall of 1946, Kenoza Lake, one of the sources of public water supply for Haverhill, developed a prolific growth of the blue-green algae, *Anabaena*. This growth exhibited the most stubborn resistance to normal copper-sulfate treatment within the experience of the writer.

Kenoza Lake has a superficial area of 200 acres, a total capacity when full of about 700 mil gal and a land drainage area of 850 acres. At times, the natural yield of the lake is augmented by pumping into it from a small reservoir, located on a brook about a mile distant. This brook drains a watershed of about 7.2 square miles. Considerable swampy area exists on the watersheds of both lake and brook.

For many years routine microscopic examinations had been made in the laboratory connected with the writer's office, in order to secure advance information as to when excessive growths of organisms might be expected and to put into effect preventive measures. The normal sampling schedule called for monthly samples during December to March, inclusive, and semi-monthly samples during the other months. This schedule was not fully lived up to in 1946—a fact which has some bearing upon the events which occurred.

Kenoza Lake samples collected during the early part of the year gave generally satisfactory results as regards content of microorganisms (see Table 1). A sample collected on June 18 showed a few *Anabaena* and a letter transmitting the results of the examination stated, "It is evident that the Kenoza Lake supply should be watched carefully, lest *Anabaena* develop more prolifically in the near future. If these organisms become too prevalent, copper-sulfate treatment of Kenoza Lake would be required."

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TABLE 1.—SUMMARY OF MICROSCOPICAL EXAMINATIONS OF SAMPLES FROM KENOZA LAKE

Date, 1946	Diatoms	*Chloro-phyceae*	*Cyano-phyceae*	*Protozoa*	Other	Total
	Standard units per milliliter					
March 6	825	30	—	—	—	855
May 13	970	115	—	—	—	1,085
May 31	740	—	—	200	—	940
June 7	1,545	200	—	180	—	1,925
June 18	350	20	50*	—	—	420
July 17	760	70	—	—	—	830
Aug. 13	235	50	180*	80	—	545
Sept. 5	140	—	—	—	—	140
Oct. 1	120	50	15*	—	—	185
Oct. 14	—	—	22,890*	—	—	22,890
Oct. 17	—	—	8,790*	—	—	8,790
Oct. 21	—	—	10,560*	—	—	10,560
Oct. 25	580	—	8,970*	—	60	9,610
Oct. 31	1,620	—	21,150*	—	—	22,770
Nov. 1†	—	—	20,000*	—	—	—
Nov. 1‡	—	—	15,000*	—	—	—
Nov. 15	3,335	—	8,130*	—	—	11,465
Nov. 19	4,140	—	13,920*	—	—	18,060
Nov. 25	1,510	—	5,850*	—	—	7,350
Dec. 6	250	50	1,800*	—	—	2,100
Dec. 16	847	30	110*	—	—	987

*Mainly *Anabaena*.
†Surface sample.
‡Sample at 10-ft depth.

The next sample, collected on July 17, showed no *Anabaena*. No sample was collected until August 13, at which time *Anabaena* were found present in moderate numbers. In view of this fact, the water department was advised, "Inasmuch as this organism is one which, if allowed to develop, will produce seriously objectionable tastes and odors, we recommend the immediate treatment of Kenoza Lake with copper sulfate at the rate of two pounds per million gallons." This advice was followed and a sample collected on September 5 showed no *Anabaena* and moderately low total counts. This sample, however, was four days from date of collection to receipt in the laboratory and, in the light of later developments, it is by no means certain that

Anabaena were totally absent. This organism grows in filaments, made up of small individual cells. These filaments are easily broken up, with the result that it becomes difficult to detect the individual cells. It is important, therefore, that microscopic examinations be made with the least delay possible after samples are collected.

No sample was collected until October 1, nearly a month having elapsed since the previous collection. Only a few *Anabaena* were found in the October 1 sample but, in view of a three-day delay in delivery of the sample and the fact that customer complaints of tastes and odors were occurring, the results of the examination led to telephoned advice for treatment of the lake with 2 lb of copper sulfate per mil gal. This advice was followed on October 8, a week after the collection of the sample. These delays evidently provided opportunity for the organisms to develop most vigorously. The copper sulfate application of October 8 was wholly ineffective, as shown by a count of nearly 23,000 standard units of *Anabaena* per ml in a sample collected on October 14 and examined the same day.

Treatment with 3 lb of copper sulfate per mil gal was carried out successively on October 15 and 19, and with 5 lb on October 21. These successive treatments were also unsuccessful in destroying the organisms and the complaints with respect to tastes and odors continued.

Chlorination of the supply was increased to the capacity of the chlorinating equipment, or to about 1 ppm, but without appreciable benefit. Laboratory tests indicated that complete removal of tastes and odors could not be brought about by chlorine, although substantial reduction could be obtained.

In order to determine whether the auxiliary supply, pumped from the brook previously referred to, was the source of the *Anabaena*, a sample of this water was examined. No *Anabaena* were found therein.

Samples collected from the lake on October 31 and November 1 showed continued high counts, with no indication that the prevalence of the organisms was being affected by the repeated doses of copper sulfate. This fact led to a recommendation that a solution of copper sulfate be applied to the surface of the lake by spray apparatus. The prior applications had been made in the customary manner by towing bags containing crystals of copper sulfate from a boat. Delay was

experienced in securing a suitable craft for setting up the spray equipment and in getting the equipment to operate satisfactorily. Finally the pond was sprayed with a dose of copper sulfate equivalent to 13½ lb per mil gal. This spraying operation required the greater part of a week, from November 9 to 13, inclusive.

Samples collected on November 15 and 19 showed continued high counts of *Anabaena* and it was not until the 25th of November that a marked reduction in count was found. The sample on this date contained no objectionable taste or odor and was the first sample since September which was satisfactory in this particular. A sample of chlorinated water collected on December 5 showed a moderately low count of organisms but an unchlorinated sample collected the following day showed a moderately high count.

In view of the continued presence of *Anabaena,* irrespective of the great improvement in the quality of the water, and in view of the approach of winter with accompanying freezing of the lake, a final dose of copper sulfate was applied on December 9 to 11. The rate of dosing was 13 lb per mil gal. The examination of a sample collected on December 16 showed that *Anabaena* had been practically eliminated and that such organisms as were found were in the process of structural disintegration.

During the course of the attempts to control the organisms, over eleven tons of copper sulfate were applied to Kenoza Lake. This was equivalent to about 10 lb of metallic copper per mil gal, or slightly over 1 ppm by weight.

The use of activated carbon, as well as superchlorination, was given careful consideration. The comparatively large volume of Kenoza Lake and the correspondingly large amounts of carbon which would be required, as well as the high concentration of organisms, appeared to render the use of carbon impractical, particularly as past experience with Kenoza Lake indicated that copper-sulfate treatment would be effective. Furthermore, activated carbon does not kill organisms and copper sulfate would have been required for this purpose in any event. Equipment for applying carbon would have had to be provided and this could not be done with any degree of promptness. These considerations indicated that it would be wiser to continue with the copper-sulfate treatment.

Looking backwards, it would seem that the first dose of copper sulfate should have been much greater than that which was used. However, so far as the writer is aware, there is no recorded instance where such large doses as were used at Haverhill have been required to destroy *Anabaena*. Hale (this JOURNAL, 1930, p. 361) reports doses of but 1 lb per mil gal to be effective in New York City reservoirs.

It is not clear why *Anabaena* developed so luxuriantly in Kenoza Lake and were so difficult to control this past fall. Weather conditions may have been a contributing cause, inasmuch as the fall was unusually dry and sunny. Table 2 shows some of the significant weather data.

TABLE 2.—METEOROLOGICAL DATA, HAVERHILL, MASS., 1946

	September	October	November
Temperature—°F.			
Mean	67.0	57.0	46.0
Deviation from normal	+3.3	+3.6	+4.3
Precipitation—inches			
Total	0.69	3.06	0.90
Deviation from normal	—2.60	+0.16	—2.17
Number of days			
Clear	14	19	12
Partly cloudy	9	8	8
Cloudy	7	4	10

The possibility of unusually favorable food conditions has been considered. A sample collected on December 16 showed but a trace of phosphates and 0.20 ppm of nitrate nitrogen. Other characteristics of the water as collected during the fall are shown in Table 3.

The above data afford no clue to indicate that any specific characteristic or quality of the water was a major factor in the undue growth of the organisms. It is obvious, however, that there was ample food material and this fact, together with the unusual amount of fair weather during the fall, may well account for the growth. The writer has no explanation for the resistance offered by the organisms to copper-sulfate treatment. Previous treatments with water of similar quality have been successful.

TABLE 3.—COLOR, CARBON-DIOXIDE CONTENT AND ALKALINITY OF KENOZA LAKE WATER

Date, 1946	Color	Free carbon dioxide	Alkalinity
		Parts per million	
Sept. 5	30	2.8	17.0
Oct. 1	45	2.6	16.0
14	50	3.5	16.5
17	45	2.7	16.5
21	35	2.0	16.0
25	40	4.5	17.5
31	40	1.5	19.0
Nov. 15	45	1.6	16.5
19	45	2.6	18.7
25	50	1.9	21.0

Table 4 summarizes the application of copper sulfate.

TABLE 4.—COPPER-SULFATE APPLICATIONS TO KENOZA LAKE

Date, 1946	Water in storage (mil gal)	Copper sulfate (lb) Total	Copper sulfate (lb) Per mil gal
Oct. 8	500	1,000	2
15	600	1,800	3
19	600	1,800	3
22	600	3,000	5
Nov. 9-13	600	8,000	13½
Dec. 9-11	550	7,100	13
		22,700	39½

A sample collected on December 16 and analyzed by the State Department of Public Health was reported as showing 0.82 ppm of copper as copper sulfate ($CuSO_4 \cdot 5H_2O$).

Certain lessons may be learned from the experience at Haverhill. First, it is important that laboratory examinations for control of microörganisms should be made at sufficiently frequent intervals to preclude undue development of objectionable organisms before detection. Second, examinations should be made as soon as possible after

sample collection. Third, copper sulfate should be used preferably as a preventive rather than a corrective measure. Fourth, promptness of application of copper sulfate, when need is indicated, is essential. Fifth, too much copper sulfate is to be preferred to too little. Sixth, in the case of surface-water supplies subject to growths of micro-organisms, particular vigilance should be exercised to forestall trouble during prolonged dry periods with more than usual sunshine.

Acknowledgment for information in connection with the above-described experience is due C. P. Driscoll, Superintendent of the Haverhill Water Works. Mr. Driscoll had the misfortune to be on the receiving end of the complaints from irate citizens during the greater part of last fall.

THE USE OF ACTIVATED CARBON IN TREATING OPEN RESERVOIRS

BY HOWARD E. BAILEY* AND JAMES STUDLEY†

[*Read December 19, 1946.*]

ACTIVATED carbon was applied to Great Sandy Bottom Pond in Pembroke as a treatment for tastes and odors, without subsequent filtration of the water. The treatment, in connection with the use of copper sulphate, was successful in eliminating tastes and odors from the joint water supply, serving 16,000 customers at a rate of about 1 mgd in the towns of Abington and Rockland, Mass.

Great Sandy Bottom Pond

Great Sandy Bottom Pond, shown in Figure 1, is the source of water supply for the two towns and lies in the town of Pembroke, about 5 miles away. Tributary to the pond is Little Sandy Pond about one-half mile above, with a cranberry bog located on the stream between the two ponds. The area of Great Sandy Bottom Pond is approximately 108 acres and the capacity 350 mil gal at high water. At the time the treatment was carried out, the capacity was approximately 300 mil gal.

The maximum depth of the pond is 23 ft at a location several hundred feet north of the intake, where the depth of water over the 24-inch cast-iron pipe is only 6 ft.

Early Troubles with Tastes and Odors

The water-supply system was established in 1885 and no trouble was experienced as of record until 1908. Again in 1927, the tastes and odors were apparently aggravated by use or misuse of chlorine for a period of two or three years. In 1934, the taste-and-odor trouble was definitely attributed to the growth of algae. There were intermittent troubles between 1938 and 1944, when the quality of the water varied from "good" to "bad".

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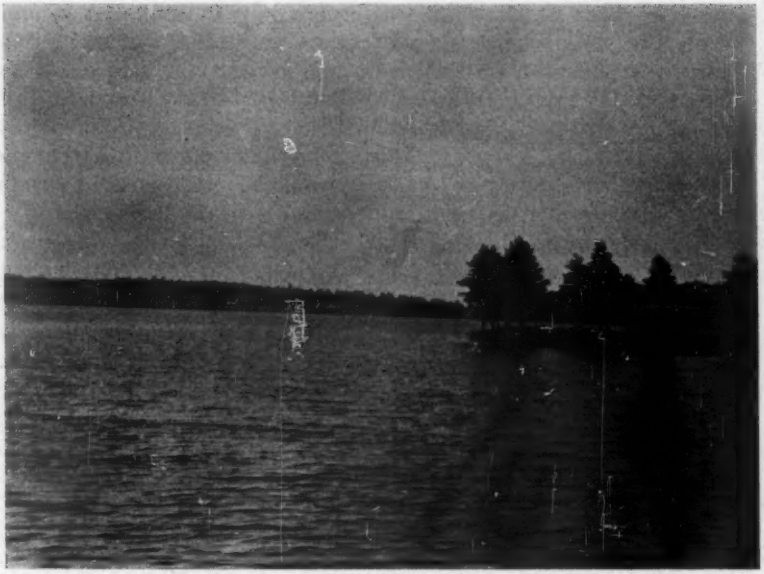

FIG. 1.—VIEW OF GREAT SANDY BOTTOM POND, PEMBROKE, MASS.

In the spring of 1944, tastes and odors which were described as "pigpen" and "fishy" were recorded. A treatment with 7,000 lb of copper sulphate, or approximately 3 ppm; made no change in the quality of the water, it is understood, but required removal of dead fish from the reservoir at a cost of $300. With the colder weather in December and the formation of ice, the tastes and odors disappeared.

In the spring of 1945, tastes and odors became prominent early in the year. In January, *staurastrum* was present to the extent of 894 areal standard units per cc. In April, 910 units of *clathrocystis* appeared in the pond water and, in the tap water at the pumping station, 2,310 units.

Treatment with Activated Carbon

Since the towns of Abington and Rockland had no other source of supply and use of copper sulphate alone was not effective, the

question arose whether activated carbon would prevent the tastes and odors, would stay in the pond or would require filtration before the water entered the distribution system. The customers apparently had no objection to "purifying charcoal", as they called it.

Upon advice from the manufacturer, copper sulphate was first applied to the open reservoir, followed in a day or two by activated carbon. The chemicals were applied from two outboard motorboats, following a route around the shore in the shallow water and the usual criss-cross pattern in the deeper areas (see Fig. 2).

Dosage

On July 24, 1945, 1,500 lb of copper sulphate, equivalent to 0.6 ppm, were applied to the pond by the usual method of dragging burlap bags filled with crystals behind an outboard motorboat. Early in the morning of July 25, a breeze was blowing over the pond, accompanied by an unpleasant odor of fish. The turbidity of the water was high and water drawn from the tap in the pumping station was distinctly unpalatable. By 1:00 P.M., 1,000 lb of activated carbon had been introduced into the pond water and a reduction of the fishy odor was observed. By 8:00 o'clock in the evening, a total of 3,000 lb of carbon had been distributed by outboard motors, corresponding with a dose of 1.25 ppm. No carbon was distributed within 100 ft of the intake.

Results

On July 26, the morning after the carbon treatment was completed, no taste, odor or turbidity was observed in the tap water at the pumping station, nor did any carbon appear.

The screens in the pump well were covered with a gelatinous mass, which was flecked with carbon particles but did not pass through the screen. During the first day after carbon treatment, the screens required washing at one-half hour intervals, where previously the screens were washed only once daily. The gelatinous clogging disappeared the next day and did not again appear when copper sulphate or carbon was used alone.

The *turbidity* decreased and disappeared within a few days.

A gelatinous mat, with a thickness of about ½ in., covered the

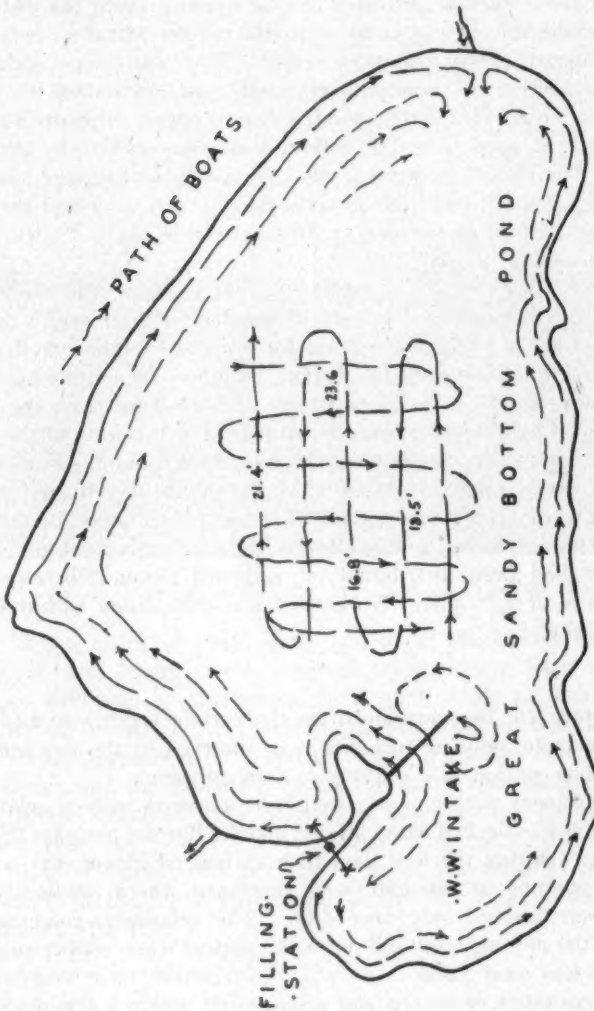

FIG. 2.—METHOD OF DISTRIBUTING ACTIVATED CARBON IN AN OPEN POND.

bottom of the pond. A microscopic analysis indicated that this was composed of amorphous matter and that no live algae were present.

The *chlorine consumption* dropped 3 to 4 lb per mil gal.

Tastes and odors disappeared all over town one week later.

Sandy Bottom Pond six months later was again clear and the sand again appeared clean beneath the shallow areas of the pond.

Recurrence of Tastes and Odors

A recurrence of tastes and odors was expected and, on August 6, 2,000 lb of activated carbon were added, when a slight taste began to appear at the pumping-station tap. On this and later occasions no attempt was made to avoid the 24-inch pipe intake in the distribution of carbon. The pond water remained without tastes and odors for the balance of the year. As a precaution, just before the ice formed, a dose of 0.4 ppm of copper sulphate was added on November 16 and 1.25 ppm of carbon on November 17 and 18. In the spring of 1946, a slight taste was evident and the standard treatment of 900 lb of copper sulphate and 3,000 lb of carbon was made on April 17 and 18.

In the fall of 1946, when the cranberry bog was drained, the water entering the pond had a color of 180 and *spirogyra* was present to the extent of 2,400 standard units. As a precaution, the usual dosages of copper sulphate and carbon were made on August 21-23.

Method of Applying Carbon

The carbon was applied in a water suspension, distributed from a filling station on the land and piped directly to the containers in the outboard motorboats. During the earlier treatments, the carbon was shoveled with scoops into a hand mixer contained in a steel oil-barrel (see Fig. 3). The complete hand-mixing and filling equipment is shown in Fig. 4. This method of hand-mixing was abandoned in favor of a two-barrel electrical mixing, the apparatus being shown in Fig. 5.

The carbon and water were introduced into the upper barrel, which contained an electrically driven cellar drainer of the centrifugal-pump type. The liquid was drawn through the bottom of the pump, mixed with the carbon in the impeller and released at a good velocity

FIG. 3.—ORIGINAL METHOD OF LOADING CARBON IN A HAND MIXER.

FIG. 4.—VIEW OF HAND-MIXING AND BOAT-LOADING EQUIPMENT.

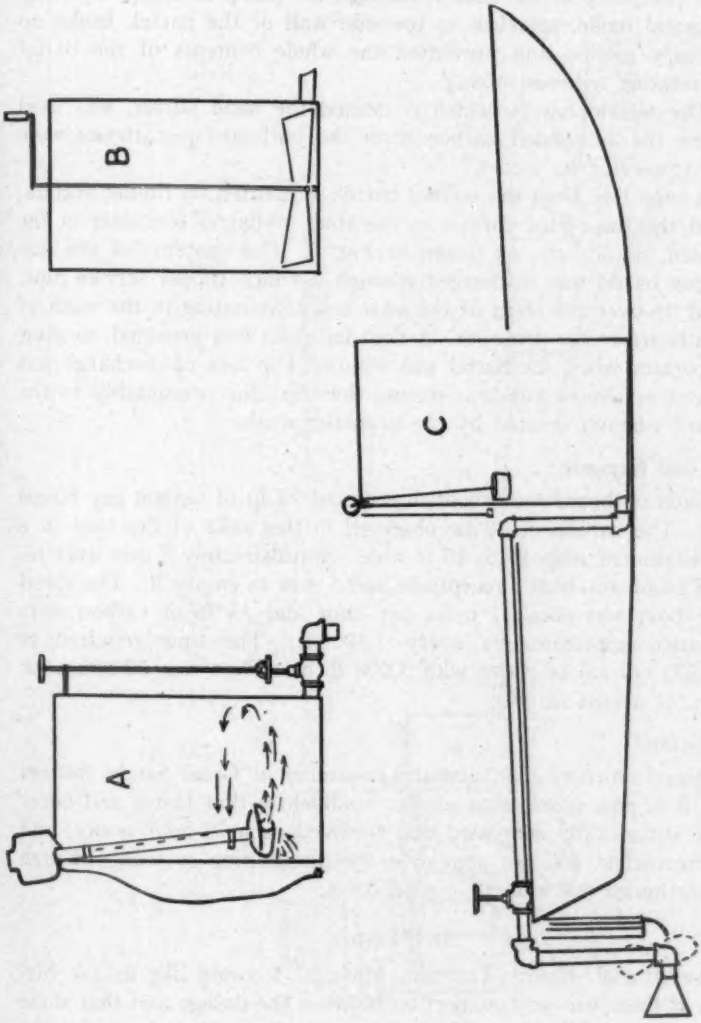

FIG. 5.—SKETCH OF CARBON-TREATMENT EQUIPMENT:
(A) Mixing Tank and Electrical Pump;
(B) Storage Tank with Agitator;
(C) Distributing Boat with Tank and Discharge Pipe.

on the periphery of the barrel, through the pump discharge opening. A diagonal baffle, attached to the side wall of the barrel, broke up the rotary motion and prevented the whole contents of the barrel from rotating without mixing.

The second barrel, which contained the hand stirrer, was used to store the suspended carbon until the outboard motorboats were ready to receive it.

A pipe line from the second barrel, supported on timber frames, carried the suspended carbon to the steel oil-barrel container in the outboard motorboats, as shown in Fig. 6. The contents of the distributing barrel was discharged through a 1-inch copper service pipe, carried up over the stern of the boat and terminating in the wash of the outboard-motor propeller. A float indicator was provided, to show the operator when the barrel was empty. The rate of discharge was observed as almost constant during the trip, due presumably to the constant vacuum created by the propeller wash.

Time and Expense

Each outboard motorboat distributed 75 lb of carbon per 50-gal barrel. The carbon could be observed in the wake of the boat in a well-distributed strip 15 to 20 ft wide. Approximately 3 min were required to fill the boat's receptacle and 5 min to empty it. The speed of the boat was about 7 miles per hour and 35 lb of carbon were distributed approximately every 1,500 ft. The time required to treat 300 mil gal of water with 3,000 lb of carbon was 10 hours for 4 men, at a cost of $40.

Conclusions

Based on more than a year's treatment of Great Sandy Bottom Pond, it is now possible to predict confidently that tastes and odors can be successfully combated and removed on a 72-hour notice, and that the carbon will not appear in the distribution system, although no filtration of the water is provided.

Discussion

Arthur C. King (Taunton, Mass.). I would like to ask Mr. Bailey if there was any connection between the dosage and that slime that he spoke of, either in the appearance of the slime or in the dis-

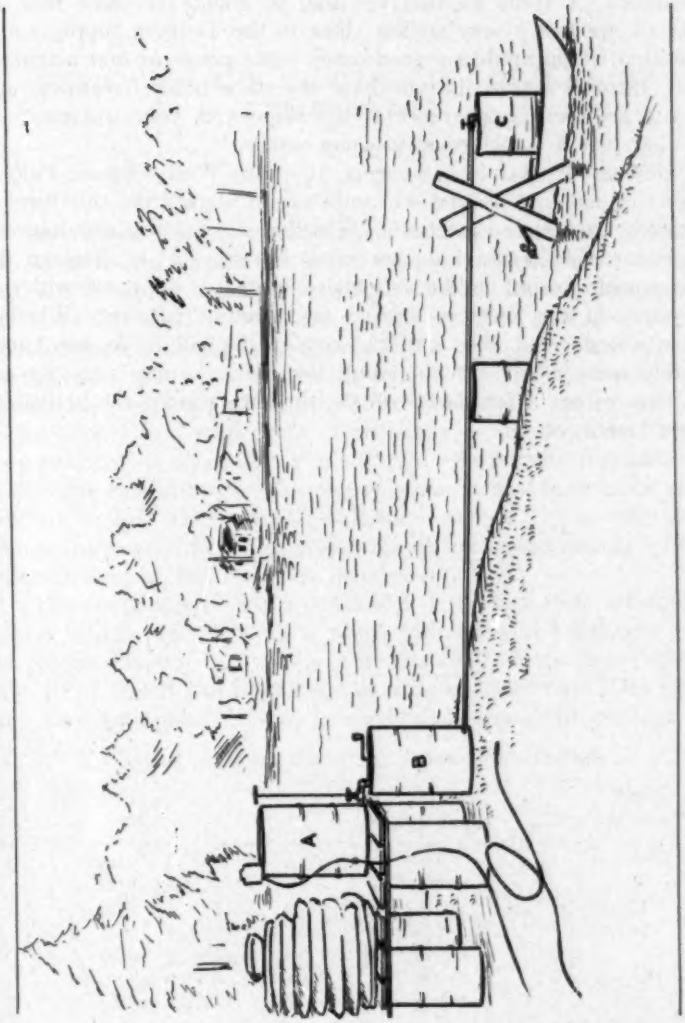

FIG. 6.—DIAGRAMMATIC SKETCH SHOWING GENERAL LAYOUT OF CARBON-TREATMENT PLANT.

appearance. I speak of that because, at about the same time he spoke of, we had a very similar slime in the Taunton supply and I know that it appeared in a good many other ponds in that neighborhood. In other words, did you have the slime before treatment and did you get the disappearance of the slime with your treatment, or was that just a coincidence, in your opinion?

HOWARD E. BAILEY. Someone from the West Virginia Pulp & Paper Co. had told us that we could expect slime from this type of treatment, so that we expected it. Whether or not it was simultaneous with some other ponds I do not know but we got no slime on the screens until we put in the activated carbon. It appeared within a few hours of the time we started the carbon treatment. Whether that was connected also with the copper sulphate I do not know. Possibly some chemist could study that out. At any rate, we got the slime within a few hours of the time we started the activated-carbon treatment.

REVIEW OF THE PROGRESS IN BREAK-POINT CHLORINATION

BY A. E. GRIFFIN*

[*Read December 19, 1946.*]

ANY process, whenever first introduced to a particular field, will be viewed with a certain amount of reserve, until sufficient time has elapsed to establish a performance record. The "break-point process" has stood this time test and the record has been good, because the users are increasing yearly and, once it has been adopted, few indeed are the cases where it has been discarded. The major change in the process has been one of nomenclature. Originally the process was known simply as "break-point chlorination"—now the term "break-point process" is used almost universally. Earlier the residuals produced were known as "break-point residuals"; now these same residuals are called "free available-chlorine residuals." The only other changes have been in the expansion of its usefulness and in a better understanding of the reactions involved.

The soundness of the original idea is perhaps best attested by its use record. In 1939 not a single operator was knowingly using the process—now it is in use in more than 370 plants (see Table 1), only 40 of which can be classed as strictly industrial. This means that the break-point process is used in at least 330 communities,

TABLE 1.—PLANTS USING THE BREAK-POINT PROCESS

Date	No. of plants	Yearly increase
January 1, 1941	33	—
January 1, 1942	72	39
January 1, 1943	93	21
January 1, 1944	123	30
January 1, 1945	178	55
December 1, 1945	261	83
December 1, 1946	370	109

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representing about 10% of the population within the United States which is served with treated water.

Progress in Usefulness

The progress made by any process should not be judged alone by the number of installations made in any given period but by its effect on society and the welfare of people in general. Chlorination has always enjoyed an enviable position in this respect and this position has expanded many-fold subsequent to the development of the break-point process.

This process was first advocated as an agent for the destruction of tastes and odors. In this it was eminently successful, except that, as with all processes designed to improve the taste and odor of water, there are limits to its use for such a purpose. In spite of any limitations it may have in this respect, the process is still used and adopted for such benefits alone.

Coagulation

It was early observed that, as soon as the chlorine application was raised to break-point proportions, the character of the floc improved sometimes to such an extent that substantial savings in the coagulant could be made. This in turn led to increased filter runs, sometimes as great as 300%. Better color removal is a direct result of improved coagulation. In some instances the chlorine bleached the color directly and thereby led to the elimination of the coagulant altogether. Such instances are, of course, relatively rare but at least one large city in the United States is using chlorine directly to remove the last vestige of color after pre-treatment with lime and alum for softening.

Iron

The removal of iron from water has presented few difficulties, although occasionally the build-up of slimes and algae on the aerators has adversely affected the efficiency of the process. These effects have been successfully overcome by producing a free available residual in the water ahead of the aerators. Naturally there are some who object to this but, contrary to previous beliefs, chlorine will not leave the water as it passes through the aerators. The odor so often preva-

lent in the vicinity of the aerators is usually composed of the chlorine compounds that must be expelled if the water is to be acceptable.

Manganese

Manganese resists removal from water to a remarkable degree. Usually it can be removed only by oxidation at a high pH-value, followed by filtration through sand. Experimental work incidental to break-point chlorination has indicated that manganese can be removed from water, without changing the pH, by chlorinating beyond the break-point.

As contrasted to the iron reactions, the oxidation of manganese is slow. For the first few hours after chlorination of a water containing manganous manganese, very little visible change takes place. Then slowly, with the first dosages beyond the break-point, the water will begin to take on some color. By the end of 24 hours, provided the samples have remained in a quiescent state, a visible floc will have made its appearance. In 48 to 72 hours the manganese will settle out. Contact periods of this length are not required in practice, because the oxidized manganese, which requires a long period of contact for removal with straight settling, will be taken out as the water passes through the filters, even with a contact period as short as 30 minutes. Baltimore is probably the largest city using chlorine for the removal of manganese, although literally scores of smaller cities are doing so, even though manganese removal is a secondary accomplishment.

Algae

The average water-works operator is continually beset with the problem of algae control. Potassium permanganate, ozone and copper sulphate have been used—with effectiveness heavily in favor of copper sulphate. As early as 1923 it was suggested that the liberal use of chlorine would kill and prevent the growth of algae.

From then on, numerous attempts were made to control algal growths in this manner, usually with inconsistent results. In each attempt, however, there were so many encouraging signs that hope was never abandoned that complete success would be obtained eventually. The tests evolved for differentiating between free available-chlorine residuals and combined available-chlorine residuals in the

development of the break-point process now appear to have been the key in this respect.

Utilization of these tests showed immediately that, when free available-chlorine residuals were produced and maintained in the water under treatment, all types of algae could be killed. This is because chlorine not only kills algae directly but, in addition, removes a large part of the food supply—namely, the ammonia nitrogen and the nitrogen in the form of nitrites.

The inconsistent results formerly obtained were usually the result of a continual reversion from one type of residual to another. In this respect the chloramine residuals quite possibly could even have stimulated the growth of certain species of algae. This is because chloramines revert to ammonia (a good algae food) as the chlorine portion is used up.

Bacteria

It was presumed from the start that, if free available chlorine would destroy tastes and odors, certainly it should be equally destructive to bacteria. This presumption has been proved over and over again, both in the laboratory and in the field, until there is little doubt but that free available residuals will destroy practically all of the bacteria that grow in water. In fact, the presumption now is that free available-chlorine residuals will inactivate viruses that may exist in water supplies.

For laboratory proof that free chlorine destroys pathogenic bacteria within a few minutes (2.0-3.0) the profession is indebted to Max Levine of Ames, Iowa, and to H. W. Streeter of the United States Public Health Service Laboratory, Cincinnati, Ohio. Lensen, Rhian and Stebbins of the Michigan Department of Health and Ridenour and Ingols of the University of Michigan have made important contributions regarding the effect of chlorine on the virus of polio. Fair and Chang of Harvard University have refuted the old idea that amoebic cysts will resist chlorine residuals of several hundred ppm by demonstrating the conditions required to produce an effective kill. These contributions were made subsequent to the development of the break-point process and it would appear they could be considered as progress resulting from the process.

New Test Methods

The new test methods for the determination of free available-chlorine residuals and combined available-chlorine residuals cannot be overlooked when considering the progress made in the break-point process. The Laux test, the ortho-tolidine-arsenite test and the amperometric titrator were perfected out of the necessity of controlling the process. With their help the process is progressing in an orderly manner. Without them we might yet be struggling with the control. In this respect, Moore, Laux, Nickel, Keineth and Connell also have developed tests to control the production and to check the existence of free available-chlorine residuals. Although the latter tests are used to only a limited degree, each has contributed to the general advance made in chlorination during the past few years.

Chlorine Requirements

The reports of "colossal" chlorine requirements to reach "break-point", as at Ottumwa, Iowa, and Anderson, Ind., in 1940—where peaks of 110 ppm and 35 ppm, respectively, were reached—may have led some operators to feel that the process required too much chlorine and, therefore, could not be used universally. That this is not the case is shown in Table 2.

This table shows that, where the water source is relatively free from pollution, as is generally the case in New England, the chlorine application to obtain a free available-chlorine residual will average 2.0 ppm. This applies also to similar supplies in other parts of North America, such as at Baltimore, Md., and Winnipeg, Canada. As the pollution in the water source increases, however, as at Olean, N. Y., Charleston, W. Va., and Frankenmuth, Mich., so will the chlorine requirements, regardless of the type of chorination employed. At Enka, N. C., the pollution is manganese, whereas at Frankenmuth the pollution is largely organic. In certain areas, such as at Bunkie, La., the pollution is in the form of sulphur and related bacteria in the ground water. The water at Bunkie, incidentally, is obtained from wells.

Summary

The record shows: 1) that "break-point chlorination" has progressed from a process with a single purpose to one of many purposes;

TABLE 2.—CHLORINE APPLIED AND RESIDUALS PRODUCED AT PLANTS USING THE BREAK-POINT PROCESS

Plant	Average chlorine applied (ppm)	Average residual at plant (ppm)
Stamford, Conn.	2.00	—
New Haven, Conn.	2.10	0.84
Lebanon, N. H.	2.00	0.35
Baltimore, Md.	3.00	0.40
Winnipeg, Canada	3.00	0.40
†Enka, N. C.	3.5	0.80
*Olean, N. Y.	3.9	0.12
Charleston, W. Va.	4.23	1.00
Tyler, Tex.	5.00	0.70
Iron Mountain, Mich.	6.00	1.00
Media, Pa.	6.00	0.20
Streator, Ill.	6.2	1.50
Bunkie, La.	10.00	0.10
Weston, W. Va.	10.00	2.10
Frankenmuth, Mich.	11.7	0.30

†Rayon plant.
*Dechlorinates with SO_2.

2) that indirectly it has been responsible for the development of several new methods for the determination of chlorine residuals, including several differential tests for distinguishing between free available-chlorine residuals and combined available-chlorine residuals; 3) that these tests have had an accelerating influence on the development of the process; 4) that the greatest single asset of the process is its ability to destroy all types of water-borne bacteria, and 5) that in general the amounts of chlorine required are not excessive.

THE WATER SYSTEM OF THE CITY OF PHILADELPHIA WITH AUGMENTED FACILITIES

BY NATHAN B. JACOBS*

[*Read January 23, 1947.*]

THE report on Philadelphia's water supply, submitted in November, 1946, by the Mayor's Water Commission, recommends the adoption of a water-supply policy, the lack of which has been the most troubling problem of Philadelphia's water supply for the past century and a half. The recommendation will be put into effect in three stages. Stage A is the immediate completion of the rehabilitation of the filtration, pumping and distribution systems and the installation of additional chemical treatment to eliminate taste and odors. Stage B will include the possible abandonment of the main channel of the Schuylkill River as a source of water supply except for reserve purposes and, in lieu thereof, the use of the Delaware River channel to supply all of the water during this stage. Stage C contemplates the obtaining of a supply of upland water for the city of Philadelphia from the upper Delaware River at a point known as Wallpack Bend, near Bushkill, Pa., and includes immediate steps to preëmpt the Wallpack Bend site for Philadelphia's use.

The present water-works system of Philadelphia is in general conformity with a program adopted in 1899, which retained the existing sources of water supply, the Schuylkill and Delaware Rivers, with the construction of filter plants, rather than the development of an upland supply.

Approximately 325 mil gal of filtered water are furnished each day to the city's 2,000,000 population and varied industries. At the present time, about half of the total supply is taken from the Schuylkill and half from the Delaware River.

Purification facilities comprise five filter plants, one of which at Torresdale takes its supply from the Delaware River and the remaining four, known as Belmont, Queen Lane, Upper Roxborough

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and Lower Roxborough plants, are supplied water from the Schuylkill. There are 10 filtered-water storage basins, 5 raw-water and 8 filtered-water pumping stations, and about 2,500 miles of distribution mains of all sizes up to 93 in. in diameter, including some 50,000 valves and over 20,000 fire hydrants. All industrial and commercial consumers and about 50% of the domestic consumers are metered.

The polluted waters of the Delaware and Schuylkill Rivers are delivered to the city's filter plants. The pollution of the streams materially exceeds the limit recommended by the U. S. Public Health Service as the permissible maximum pollution of raw water supplied to rapid-sand filter plants of conventional design for purification and subsequent use by inter-state carriers.

The pollution of the Schuylkill is due, in most part, to the discharge of sewage, sewage-treatment effluents and industrial wastes from the many municipalities and industries along the river and its various tributaries above Philadelphia; and to the discharge of acid mine drainage and finely divided waste coal from culm piles, anthracite mines and collieries, scattered throughout the upper portions of the watershed.

Sanitary sewage in the areas of Philadelphia tributary to the Schuylkill is collected by a separate system of sewers, which discharge into the river below the Fairmount Dam. Some pollution of necessity arises within the city limits, due to discharge of storm-water drains, which carry the run-off from streets and private properties during periods of storm.

The discharge of sewage is the basic cause for the high organic and bacterial content of the river water. The industrial-waste discharges contribute largely to the recurring objectionable tastes in the filtered water and also render the raw water more difficult of treatment. Hardness is primarily due to the interaction between the acid mine drainage and the limestone formations in the bed of the upper river and some of its tributaries. The culm and finely divided waste coal are the prime cause of the extensive and troublesome silting above the Fairmount Dam but have little sanitary significance as concerns treatment of the water.

The pollution of the Delaware is due not only to discharge of sewage and sewage-treatment plant effluents by municipalities situ-

ated along the river and its tributaries above Philadelphia, but also to the effluent from the city's Northeast sewage-treatment plant, situated at Wheatsheaf Lane, which twice each day is carried upstream by flood tide approximately one mile above the raw-water intake of the Torresdale filtration plant.

Industrial-waste discharges at the present time are a contributing factor in the pollution of the Delaware River at Torresdale. This is manifest by the occasional presence of taste-producing material in the water but, as the volume of upland water available for dilution is generally greater than in the Schuylkill and as the total volume of such discharges is less, the tastes are not as noticeable nor of as frequent occurrence as in the Schuylkill. Discharges of culm and mine drainage do not enter into the pollution of the Delaware River at Torresdale.

Both the Schuylkill and Delaware are now polluted to an extent greatly exceeding the maximum recommended as permissible for raw-water supplies to be purified by standard rapid-sand filtration. Such filtration must, therefore, be supplemented by auxiliary treatment to an extent much greater than recognized as customary in treatment of waters carrying the maximum permissible degree of pollution. Slow-sand filtration as recommended constitutes such a method.

Water from the Schuylkill is pumped by the Belmont and Queen Lane stations, located above Fairmount Dam, to large sedimentation basins at filter plants of the same designation, and by the Shawmont pumping station, located above Flat Rock Dam, to a sedimentation basin at the Lower Roxborough filter plant. From here, a portion of the settled water is re-pumped to the Upper Roxborough filter plant. The Belmont and Shawmont stations are completely electrified, while the pumps at Queen Lane station are driven by high-pressure steam turbines.

Water from the Delaware River is taken from the tidal basin at Torresdale by a completely electrified, low-head pumping station and discharged to the adjacent filter plant.

All raw-water pumping stations have been subject to partial modernization and the present water-works improvement program includes further rehabilitation, particularly with respect to the facili-

ties at Queen Lane, which are now in process of electrification. Shawmont station ultimately will be abandoned.

The five filter plants in the present system provide varying types of treatment, ranging from single filtration by slow-sand filters at the two Roxborough plants to double filtration, including supplementary chemical treatment, at the Torresdale plant. The sedimentation basins preceding the filters of the four Schuylkill filter plants were originally used for distribution of raw water to the consumers, prior to construction of the present filter plants.

The Upper Roxborough plant, having a nominal capacity of 20 mgd, and the Lower Roxborough plant, having a nominal capacity of 10 mgd, employ single filtration by slow-sand filters and chlorination of the filtered water. Both plants will be abandoned and the areas which they now serve will be supplied from the Queen Lane filter plant.

The Queen Lane filter plant has a nominal capacity of 100 mgd, of which 50% is twice filtered, first through pre-filters of the rapid-sand type and then through slow-sand filters. The remainder of the supply from this plant is pre-chlorinated, clarified by sedimentation with alum as a coagulant, passed through rapid-sand filters and combined with the output of the slow-sand filters prior to final chlorination and distribution. It is proposed to increase this plant to 120-mgd capacity under the current improvement program and provide modern coagulation, together with facilities for double filtration of all water.

The Belmont filter plant has an aggregate capacity of 70 mgd, of which 40 mil gal are purified by slow-sand filters and the remainder by rapid-sand filters. Alum and chlorine are applied ahead of the rapid-sand filters and all water is chlorinated after filtration. It is proposed to reconstruct this plant to provide for double filtration of all water.

The Torresdale filter plant has a capacity of 200 mgd and provides double filtration of the entire output. Auxiliary treatment consists of coagulation of the water in the tidal basin before pumping to the pre-filters. Provision for application of chlorine is made between the tidal basin and the pre-filters. The water is again chlorinated at the clear well before distribution.

The existing purification works, judged by the standards of the period during which they were built, were well designed and constructed. However, the pollution of the rivers during the past 40 years has increased with the growth of population and industry on the watersheds.

Under present conditions, there are certain weaknesses in the system and inherent deficiencies in the extent of the facilities provided. For example, slow-sand filtration preceded by sedimentation alone, as now employed at the Belmont and Roxborough plants, does not provide adequate treatment for water from the Schuylkill River. The pre-filters at the Queen Lane and Torresdale plants are now obsolete in design and are not adapted to produce consistent results. The coagulating basins and mixing chambers ahead of the rapid-sand filters at the Queen Lane and Belmont plants are inadequate in capacity and design to condition the raw water properly for application to the filters. The arrangement at Torresdale, which provides for low-lift pumping from the sedimentation basin in the river directly to the plant, makes it impossible to apply a suitably coagulated water to the pre-filters.

The rehabilitation of the purification works, as planned, includes the construction of modern-type coagulation and sedimentation basins and improved chemical and mixing facilities, so as to provide adequately for pre-treatment of the water prior to filtration; the construction of modern rapid-sand filter plants, to replace the somewhat antiquated and inefficient roughing filters; the adoption of a policy of double filtration for the entire supply, utilizing the existing slow-sand filters as final or polishing units for the effluent from the rapid-sand filters, and the installation of suitable equipment and treatment facilities for overcoming taste and odor difficulties when they occur.

The entire water supply of the city must be pumped in order to develop pressures adequate for distribution.

In the case of the Schuylkill water, this pumping occurs before filtration and permits most of the filtered water to be delivered to consumers by gravity, except for about 20% of the combined filtered-water production at the Belmont-Queen Lane-Roxborough facilities, which is re-pumped by a number of booster stations to points in West and Northwest Philadelphia. Some of the booster stations are cur-

rently planned for rehabilitation or reconstruction in new locations and a few new stations will be added to the system.

All water from the Delaware River channel, after filtration at Torresdale, is distributed by direct pumping at the Torresdale and Lardner's Point pumping stations. The former station handles about 5% of the total filtered Delaware supply and its capacity will soon be incorporated into the new, 212-mgd, Torresdale filtered-water pumping station, now under construction at this location. The Lardner's Point station has been operated for many years by steam power, with a combination of triple-expansion engines and steam turbines. This equipment is being replaced by electric-motor-driven centrifugal pumping units, with an aggregate capacity of 210 mgd, of which one 40-mgd and two 25-mgd units are now in service. When completed, the station will operate with the new Torresdale filtered-water pumping station on a complementary basis. Both stations will continue to serve that area generally east of Broad Street with filtered Delaware River water.

The total capacity of reservoirs for storage and distribution of filtered water approximates 890 mil gal, including about 190 mil gal of clear-well capacity at the various fiter plants. With the exception of the 50-mil gal clear well at Torresdale, all filtered-water storage is located at elevations which permit gravity distribution. When the Roxborough filter plants are abandoned, the present sedimentation basin will be converted to filtered-water storage, which will increase filtered-water storage capacity to 1,037 mil gal, or about 3-1/3 days' supply, on the basis of current requirements. The only new storage facilities planned for construction at this time are elevated tanks with capacities of one million gallons each, to meet peak demands in the Chestnut Hill, Fox Chase and Somerton areas.

The distribution system is subject to constant extension and reinforcements, with the growth of the city and changes in location of areas of concentrated demand. The water-works improvement program contemplates considerable main construction to meet such situations, as well as general minor installations throughout the city to improve service to consumers. The water-works improvements currently being constructed were originally planned in 1940 as a general program of rehabilitation, to utilize advances in the science of

water purification for coping with increased pollution of raw-water supply and to provide capacity to meet the needs of the city to about 1965, but this program was deferred due to the defense program and the war. Temporary measures were taken to continue an adequate supply of water to take care of the additional load put on the system by the war industries.

In 1940, at the same time that the water-works rehabilitation program was started, a contract was made with Ozone Processes, Inc., to experiment with the use of ozone to eliminate taste and odors in the raw water, some of which were accentuated by the use of chlorine. As a result of these experiments and the desire of the Water Commission and City Council to try out the use of ozone on a plant scale, a contract has just been awarded for furnishing and installing a complete ozonation system, to deliver 1,250 lb. of ozone per day adequately and satisfactorily and to apply the same to an average quantity of 36 mgd of the raw water, which will be subsequently filtered through the rapid-sand filters at the Belmont plant.

The reports of the Board of Consulting Engineers to the Philadelphia Water Commissions have dealt mostly with that section of the Water Commission's report designated as Stage C, which contemplates going to upland sources.

Preliminary estimates were made of the cost of each of these projects, as follows:

Yardley-Wallpack Bend Project	$152,500,000
Delaware River-Yardley Project	$188,730,000
Upper Lehigh River Basin Project (331 mgd)	$240,000,000
Delaware River Project (Wallpack Bend)	$269,400,000
Lehigh-Pocono Project	$376,720,000
Upper Delaware River Basin Tributaries Project	$380,250,000

After the Water Commission had definitely expressed its selection of the Delaware River (Wallpack Bend) site, with the conduits leading to the regulating reservoir at Warrington, instead of utilizing the bed of the river between Wallpack Bend and either Easton or Yardley, a more detailed study was made of this project and the estimated cost of all facilities—including land acquisition and damages, purification facilities and distribution connections within the city—was $284,588,000.

The Delaware River project (Wallpack Bend) provides for an average of 500 mgd of water and involves the construction of a dam in the northeasterly course of the great "S" curve of the Upper Delaware River at Wallpack Bend, near Bushkill, Pa.; a regulating reservoir in the vicinity of Warrington, Bucks County, and tunnels to deliver water by gravity to the regulating reservoir and thence to the site of the present Queen Lane filter plant for purification.

The volume of water available from Wallpack Bend Reservoir would be adequate for the estimated future needs of the city.

The water would be much softer than that now obtained from the Schuylkill and contamination would be much less than that of present raw water supplied to the city's filter plants. It could be adequately purified by sedimentation and filtration. About 85% of the total water requirements of the city could be delivered to the existing filter plants by gravity and pressures for pumping the water from Queen Lane filter plant to Belmont and Roxborough would be much less than now required for furnishing raw Schuylkill water to these three plants. Delivery of the raw-water supply through tunnels, deep in ledge rock, constitutes a greater safeguard against interruption of supply than delivery through force mains having shallow coverage, such as those now in use.

The chief disadvantage of the Delaware River project is its distance from Philadelphia—about 80 miles of tunnel being required—which is reflected in its relative cost. The inter-state character of the project will necessitate a treaty between Pennsylvania and New Jersey, and possibly New York. It may also require Federal action and, therefore, it is necessary to proceed by stages to bring about this ultimate development.

Discussion

Arthur L. Shaw (Boston, Mass.). Are there any questions? Someone must have some questions about the problem of changing steam to electricity or vice versa. Anyone who has operated steam stations cannot help having a little feeling of wistfulness, when he sees some of these fine, old high-duty engines, that have given 40 or 50 years of service, torn down but of course an end has to come to all things.

A MEMBER. Isn't 180 gal per capita per day rather high, or is there a large industrial demand?

NATHAN B. JACOBS. There is a large industrial demand. The city is about half metered, that is, domestic consumers. The ones who are not metered are generally the smaller domestic consumers, where it is not felt that a great saving will be effected, but there is no doubt that, if they were to complete the metering, they could reduce that 180 gal per capita down somewhere near where New York, Boston, Baltimore or Cleveland are, which is nearer to 150. There is no doubt that they can. They have done a great deal on the reduction of waste and leakage.

With all of the war demands, the totals at Philadelphia were not much more than they were prior to the war. The average use of water increased from about 130 mil gal to about 360 mil gal in 1944, which was the big year.

ARTHUR L. SHAW. Do you happen to know, Mr. Jacobs, how the pollution of the present Philadelphia sources compares with the pollution of the Merrimack?

NATHAN B. JACOBS. No, I don't know, Mr. Shaw, what the conditions are on the Merrimack. I do know that about 15 years ago, when we were making a comparative study, we said that the Lawrence conditions could easily be taken care of, because Philadelphia was taking care of water and making it acceptable for drinking purposes from a pathogenic point of view. I believe at that time Chester was using it. Of course, theirs is much worse than Philadelphia's. I believe that Youngstown was using the Mahoning River and the other notable example was Albany. But both Albany and Youngstown have abandoned their nearby sources and have gone to the mountains, so that Philadelphia and Chester, I guess, are the only ones that are comparable to Lawrence, Mass.

THE CONSTRUCTION OF WATER-WORKS STRUCTURES

BY JAMES E. MUNROE*

[*Presented January 23, 1947.*]

As A builder of a great many types of water-works structures, such as all types of intakes, high-service concrete reservoirs, rapid- and slow-sand water-purification plants, and all the appurtenant structures, and contractor for the modernization of steam plants and obsolete filtration plants, I will attempt to give you, gentlemen, some of the problems that a contractor has, from the time that he receives a set of plans as a bidder until he has brought that set of plans into an actual operating facility.

Usually our organization, upon receiving a set of plans, makes a very careful study of the plans for at least two days. We try to get firmly fixed in our minds just what the engineers are trying to accomplish. In that way it gives us a mental picture of what we are going to arrive at and tends to more accurate estimates. During the latter stages of that study we try to determine what type of soil we have to contend with, as a majority of such structures are in ground water and below the ground-water surface. The information that the engineers give us on some of the plans varies from quite a complete analysis of the underground earth or rock structures to no information at all. In fact, we have figured a good many jobs where there is nothing whatsoever on the plans to indicate the type or nature of soil that we are going to have to contend with, nor is there any information with respect to the ground-water elevation.

We have often had to test the soil in several places, prior to the time that we submit our bid, and in most cases we have to go and look at the site with a shovel in our possession, dig into the sides of the test pits, and determine what is actually there. Other plans will show a few wash borings.

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Water Problems

The handling of the water on such a project is one of the major gambles and the experienced contractor is bound to try to gather all the information possible. If the borings show that the ground is porous and of a gravelly nature, sometimes that information is sufficient to allow us to establish the fact that we can dry up the excavation with a well-point system. However, in some of our river valleys, such as the Connecticut River valley, the Merrimack River valley and a good many others, if the structure is down near the river bank, you have to contend with silt, fine sand and clay. If the engineer has wash borings taken and they are labeled "fine sand," those borings are not worth the time it took to put them on paper. The fine sand can run anywhere from sand of a size which we use for concrete, brick mortar or plastering down to sand which would pass through 200 meshes to the inch—fine as flour—with all the characteristics of being Portland cement, and can still be classified and named as fine sand.

Inasmuch as the success of a well-point system is totally dependent upon the flow of the water through the sand, we must have some information enabling us to determine what the break-off point is in this fine material, in order to determine whether we can use well points or whether we have to go into a different method of handling the ground water. Therefore, it is my contention that the engineers, for the sake of protection to their clients and for the sake of avoiding trouble, should make it a point to have core borings made by a reputable concern and have those boring samples contained in jars and labeled for the inspection of the contractor, so that he can make up his mind as to which is the best or most economical method of unwatering the excavation.

Sometimes the engineer feels that the cost of core borings is too great. I will venture to say that the lack of information has a tendency for the experienced contractor to figure the worst conditions, so that he will not lose money, but with the information given by the core borings, his price would probably have been more economical and the client would have saved money.

It is my recommendation that core borings should be taken at distances not exceeding 50 ft on centers, both ways, over the area

of the structures, particularly if the structures are in a water-bearing material.

To give you some idea as to the problems of handling ground water and some of the ways we have handled it in the various projects we have undertaken, I might illustrate through some examples of different projects we have completed. For instance, about 11 years ago at the Norwood, Mass., water treatment plant, the clear well in that plant was in 6 ft of ground water. It was in a good gravel stratum. At the time of estimating that particular job, there was no question but that it was a simple procedure of installing the well-point system and unwatering the excavation without trouble.

We set this system up, completed its installation about 4 o'clock in the afternoon, put the pumps into operation and pumped a full 6-inch stream of water until 7 o'clock the following morning. At 7:30 we made an observation and found we had drawn the water down 12 ft. Therefore we were able to idle the pump and maintain the ground-water elevation at any particular grade we desired, by controlling the speed of the pump engine.

Another opportunity to use the well-point system to good advantage on that particular job was that after the reinforcing steel had been placed, being able to control the water to any grade that we desired, we left instructions with the pump man, who was there during the night, to shut off the pump, allow the water to come up 2 or 3 in. over the surface of the sub-base, and then start the pump and lower it about 6 in. below that base, and to keep alternating those particular procedures until the next morning. In that way we were able to use the ground water as a means of solidifying the gravel base, so that we would have a good, firm base on which to place the concrete floor slab.

On another project, the treatment plant was located in a peat swamp. This happened to be a plant where little or no information as to ground conditions was given on the plans. The only way you could find out what conditions might be like was by going and looking at the site. Most of the natives contended that every fall and every spring there was anywhere from one to two feet of water over that particular swamp. The area to be covered by the proposed structures and the earth fill that was to be placed around and between the various structures was approximately three acres.

On the plans there was an elevation given for the bottom of the tanks, but there was no indication whatsoever as to what the nature of the material was that we were going to encounter, when we got to that elevation. There was a provision in the specifications that, in case an unstable condition was found at that elevation, payments for excavation below grade and for backfill would be made. That was perfectly all right in a dry excavation, but the farther you might have to go down in wet ground, the more costs would be involved in the handling of the water, as the water was very close to the surface of the swamp.

On that particular job it was necessary to wade around in the swamp with hip boots on and, with a steel rod, prod around through the peat and try to determine how much peat there was there and what the nature of the material was underneath the peat. It so happened that it was a good coarse gravel.

Well points would have worked very well in that particular gravel but, due to the fact that the area was so large, it was impractical to go into the operation of a well-point system, both on account of the equipment that would be necessary and on account of the length of time that would be required to build all of the structures that were in the water.

In estimating that particular project, we had every reason to believe that we could build a couple of gravel pump-wells to unwater the entire area, so we proceeded to excavate around the rim of the job site, building a dike with the peat to give us some protection against the flood waters. We brought in a certain amount of fill and backed up the peat dike, to give us further protection against the flood waters. During this operation there was a slight pitch to the gravel surface and we were able to keep reasonably dry by the use of surface pumping.

As soon as all the peat was completely stripped, we dug two pits, 8 ft square, using wood sheathing down 10 ft below the surface of the ground. We got hold of some oil drums, punched the sides full of holes, stood them up on end, backfilled the pits with crushed stone, and withdrew the sheathing, creating more or less of a rather crude gravel-packed well. By pumping with an ordinary contractor's pump, we were able, through the entire progress of the construction, to keep the water level down in that particular material.

When getting into finer material, it is necessary, of course, to make a pretty careful study of just what you have, and that is why it is absolutely necessary to have core borings, to be able to judge what you can do with that particular material. We can dry up fairly fine material by means of well points. When we get into this fine material, that to the average layman would look similar to clay, although it might not be clay, it becomes impossible to operate a well-point system and successfully dry out the excavation.

In this type of material, we usually have to go into a sheathed cofferdam and, if the pressures and water levels are high enough, it necessitates a steel sheet cofferdam, with heavy bracing. If, however, we have to go down to any great depth, in perhaps 15 or 20 ft of water, and the lower part of our excavation is in the fine material, we are able to relieve some of the pressure on our cofferdam by taking off sometimes as much as half or slightly more of the head of water that we have to contend with, by means of well points or open pumping.

The indication on the boring plan of fine sand makes us realize that we have the possibility of a source of considerable trouble before we are able to get to sub-grade and successfully get in the base concrete. For instance, on one particular job on the Merrimack River, the borings showed fine sand and from observation it looked as though we could make it all right with well points. The structure was right on the edge of the river. We put in a sheet-steel cofferdam, to keep the river out of the excavation, dug down to water level and installed a well-point system.

However, the material was of such a nature that some fines in it would move with the water and it was necessary for us to install twice as many well points as we anticipated. In other words, instead of putting them in 4 ft on centers, we had to cut the spacing down to 2 ft. We also had to add a supplementary vacuum pump on the well-point pump and we were pumping with 27 in. of vacuum on that particular system. We were literally squeezing the water out of the material and yet when we got down near the elevation of our sub-base, we found that we still had a wet condition in which to work.

Through prolonged pumping it might have been possible finally to dry up the excavation but we decided it was cheaper to drive a

low wooden cofferdam from the water elevation as it existed at that point. We had to go down some 4 ft to sub-grade and proceeded to excavate the soft material that was there, to a point one foot below grade. We backfilled with crushed rock to sub-grade, thus obtaining a stable bottom and also a bottom which we would be able to keep well drained, so that, when the concrete floor was placed, the water pressure would not break up through the floor before the final set of the concrete had taken place.

Other instances have occurred where we found that we had clay to contend with, right from the water surface down, and we have had the opportunity of battling our way down through 30 ft of water. In such a case, the experienced contractor knows that he has a condition which is going to give him a few sleepless nights and that he is going to do some real fighting before he gets down to the bottom.

We built a sheet-steel cofferdam in this particular location. When we got down to grade, 30 ft below the ground-water level, we found a bottom down there that was really alive. Most persons would call it "quicksand." Quicksand is simply sand that is held in suspension, due to a hydrostatic head. On a low head a very fine sand will become "quick" but, if you get a hydrostatic head high enough, you will find that pea stone will also act like quicksand. It is simply a matter of relieving that hydrostatic head to stop the quicksand condition that you have at the bottom of your excavation.

However, in this particular 30-foot cofferdam, we were unable to relieve the hydrostatic head by any method at all. It was necessary for us to go down approximately 4-5 ft below our normal depth of excavation, by starting in one corner of the cofferdam and excavating the 4 or 5 ft, dumping in truckloads of crushed rock, then progressively excavating across the cofferdam, and continuing to push this heavy bed of crushed rock across the cofferdam, as the excavation proceeded. This finally gave us a reasonably stable bottom. We knew that, as long as we could keep this material confined until the structure was placed upon it and the hydrostatic head was relieved, the bottom would be perfectly satisfactory for the structure itself.

In order to be sure that there was no possible chance for the fines to work their way up through the gravel bed that we had placed, while we were putting in the concrete floor and coming up with

the 30-foot walls, to get above the ground water, we placed tile drains within this gravel bed, brought them over to several sumps, and pumped the water out with the usual type of contractor's centrifugal pumps.

That particular basin is just an example of a really tough battle that a contractor might have in his experience in handling various types of conditions where he encounters water.

One may contend that, with a quicksand bottom, the foundation might be unsuitable to hold the structure. However, I might stress a point that may be interesting to some of you men. When we get our structures, although they are made of concrete and steel, up above the water line, we, of course, want to relieve ourselves of the pumping cost, because that is quite a considerable item. When we get up above the water line, we wish to stop the pumping and allow the water to come up but, before we do that, it is absolutely necessary that the contractor compute the flotation point of the structure. Usually he will find that a structure, simply brought up above the water line, does not have sufficient weight to overcome the buoyancy caused by the volume of water displaced and, believe it or not, a large portion of these structures, when brought up just to the water line, will actually float. The only way we keep them down is to fill them with pre-determined amounts of water to offset the flotation.

In fact, in the case of one structure that we built, after making the calculations of the flotation point, we found that the entire structure, when completed, would be only 5% more than the buoyancy point, discounting entirely the ground friction that might be caused by the backfill. You can readily see that you are often building a structure that may have less weight than the water that it displaces and, if so, it will come up, irrespective of whether it is built of concrete, steel or wood.

I might say that the cost of operation of any average well-point system is approximately $200 a day, due to the fact that it is necessary to maintain men to watch the pumps over the period of 24 hours a day, 7 days a week, for the duration of the unwatering operation. However, even with that expense, it is usually more economical to use well points in lieu of a cofferdam, if the material is such that it will allow the use of well points. That is where the core boring will

give the experienced contractor the opportunity to estimate the well points, when he finds that they can be used, as against estimating a cofferdam, and perhaps a cofferdam and well points, when he is in doubt.

Concrete Structures

So much for the water problem. I will go along a little bit farther and give you some idea as to what we have to do in building watertight concrete structures with the use of nothing but good concrete.

I am very much against the use of any admixtures in concrete. I believe that it is unnecessary and I have proven it to myself. I believe several of the engineers are thoroughly convinced that concrete, properly proportioned, can be made absolutely watertight, if a suitable construction joint is provided in the structure. With the proper proportions you can get an absolutely dense concrete wall and floor and keep the water out.

As for the type of construction joint at the floors and the vertical construction joints in the walls, at any place below ground-water level or in structures where there is water on one side of the wall, as in clear wells and in high-service reservoirs on dry ground, we have found that a most satisfactory job can be done by the use of a steel plate, either ⅛-inch or ¼-inch in thickness, and either 6 or 8 inches in width.

Some 10 or 15 years ago, the common practice was the use of 16- or 20-ounce copper sheets for water stops. This copper was extremely expensive. It was costly to install and in most cases failed to do the job. When the contractor poured his concrete into his forms, the concrete hit the copper and bent it over, and it actually was a total loss as a water stop. The use of this steel plate, we have found, is cheaper than copper. It is much less expensive to put in place; it is stiff and rigid, and it will not be knocked down or damaged when the concrete hits it. If the engineers want to go far enough, they can use the value of the steel plate in their shear calculations.

It is necessary at the present time, with the high cost of construction, that the concrete design on these structures be simplified just as much as possible. Back in the days when we were paying $1.25 an hour for carpenters and we were able to get plywood for

approximately $100 a thousand, on the average form work such as clear-wells, and so forth, a contractor could base his estimate on 20 cents a square foot of contact surface. Today we are right back in the horse-and-buggy days, as far as material for forms is concerned. We cannot purchase plywood at any price, not even in the black market. We have to go back to the use of boards. Instead of paying $1.00 to $1.25 an hour for carpenters, we are paying $1.75. Instead of 50 cents an hour for common labor, it is $1.00 and $1.25. Therefore, our form costs are at least 2½ times what they were in 1939. It is absolutely necessary, in order to keep the cost down, that the engineers simplify the construction in regard to the form work as much as possible.

I recall one particular plant that we bid on, in which the engineer apparently had considerable time on his hands. He had so carefully estimated the requirements of his steel and concrete that he battered the outside face of a circular tank. He saved approximately $200 worth of concrete; but in the saving of that $200 worth of concrete he increased the form costs approximately $1,000.

The battering of a concrete wall that is straight adds little or nothing to the cost but the minute that you attempt to batter a circular wall, you really start to raise Cain with the cost. When you strip your forms, the lumber is nothing but kindling. It has no value for further use.

In the construction of concrete tanks that are to hold water, it has been the practice in years gone by—and even up to the present time—not to cover them with earth or other protection to guard against the elements. The reason, in most cases, was to save money. Up here in New England, we find that concrete is perhaps one of the best structural materials we have but, as an architectural material, it is a total loss and it should be exposed to the weather only when absolutely necessary.

In the construction of settling basins that are partially out of the ground, we have found after the first year a crack that appears around the entire structure, approximately 6 to 8 inches above the ground. It is impossible for the engineer to put enough steel in the tank walls to prevent that crack. On the inside of the wall, where the water would be up against it, you would probably have a temperature

variation of 10° over the course of 12 months. That part of the outside face of the concrete wall that is below ground might possibly have a maximum variation of 30° and that part of the concrete face that is exposed to the weather here in New England might be subjected to a temperature range of 120°. Any of you gentlemen who have any idea of the problems of designing concrete know that it is practically impossible to overcome such stresses and it is necessary that the concrete be protected by earth or other material.

The cheapest method, of course, provided the surrounding land conditions allow it, is to cover this structure with earth. Occasionally you will find that the filter plant or the structure is laid out in such a way that it is impossible to cover the structure with earth, due to space limitations. In that case, some provision should be made to face up the concrete wall with a brick facing or some other suitable protection, so that the concrete is not exposed to the weather.

The first indication I have had that steps are being taken along this line is on the proposed addition to the filter plant at Burlington, Vt. I think that the engineers should be congratulated on their foresight in this particular case.

About ten years ago we built a filter plant in the northern part of New York State and I was quite proud of the concrete job that we did there. I was back there last fall and, on the concrete walls that were dry on the inside face, you could still see the grain marks of the plywood and the walls appeared to be just as good as the day they were built. But when we got around to the settling basin, which is about ¾ out of the ground, I was ashamed to have my name on the bronze plaque. In fact, if I had had any idea I could have gotten away with it, I would have removed the bronze plaque from the building, because that settling basin is a total wreck. The concrete has spalled off to a point of pretty nearly exposing the reinforcing steel.

I looked up the Superintendent of Public Works. He happened to be the same man who was superintendent when we built the plant. I told him it was absolutely necessary that he get funds appropriated to gunite the exterior of the tank, to protect the rods, and that the city should acquire adjacent land and proceed to backfill and cover the tank with earth. I warned him that, if they failed to do this, within 2 or 3 years the tank would collapse.

A suitable brick facing, with an air space of 2-4 in. or more, would aid materially in preventing this weather action that we get here in these northern climates.

Brick Masonry

In regard to the construction of the plants themselves above the concrete work—we have done our best to try and analyze what has been wrong with the masonry work that has been built in the past and what the reason is that we have so many leaky buildings. We have come to the conclusion that the brick mortar has been the primary cause of the leaky buildings and of the efflorescence on the brick walls—the objectionable salts which appear on the face of brick buildings during certain climatic conditions.

Some 20 to 25 years ago, the Portland Cement Association sold the engineers and architects of this country a "bill of goods," to the effect that it was highly necessary to eliminate a large portion of the lime that the construction industry was using in the brick mortar and to specify 10% lime mortar. In other words, they actually recommended sand and cement mortar with 10% lime.

Back in the days of my father and of some of you older gentlemen, a lot of the brick structures that are still standing were built with nothing but straight lime-and-sand brick mortar. No cement was used at all and I venture to say that not one of those buildings ever leaked through the brick walls. The 10% lime mortar is so hard that there is absolutely no elasticity to it. On buildings in which the bricks are hard burned or of a water-struck or pressed-brick nature, after the cycle of temperatures that we get in a period of 12 months, you will find that all you have is a brick wall standing there, that has actually torn itself all apart. Every brick in that wall will have broken away from the mortar on one of the two ends and either on the top or the bottom. You could put a razor blade in, out of sight, in the cracks that are there at each joint.

The Portland Cement Association, at that time, also sold the engineers and architects the idea of slushing the brick walls solidly with mortar. When the walls opened up, due to temperature variations, they simply opened up channels right through the brick work. In a good driving rain the water just poured through the brick wall

and ran down the inside face of the wall.

Our contention is that there never has been a brick wall properly built, that has failed because of the brick mortar. There is absolutely no reason for making the brick mortar any stronger than the brick, because the brick would fail, if the wall were overloaded, before the mortar would.

Our recommendation in the construction of brick-masonry walls is that two parts of lime to one part of cement by volume be used. There is hardly any necessity for specifying the amount of sand needed, because the brick masons must have a workable mortar to work with. If the mason's tender should happen to get a little too much sand in the brick mortar, it becomes brittle and bothers the bricklayer in the handling of it, and he will immediately bawl out the tender. If the tender doesn't get enough sand in it, it will become too sticky and the bricklayer cannot get it off his trowel. We mix most of our mortar by machine and the control of the sand is in the hands of the bricklayers, and they will raise Cain if the mortar is not workable.

In the construction of a 12-inch exterior brick wall, the outside course should be vaulted. The inside 8 in. of brickwork can be pargeted with a plaster coat of mortar on the outside face and at least a one-inch air space can be left between the 8-inch section and the outside course of the brickwork. The vertical brick courses themselves could be tied together, either by means of galvanized metal wall-ties, or by what we call a Flemish header. Headers should go in every fifth course of brick in height.

Some ten years ago we had to break through a section of an old brick wall of a pumping station, in which we had to install a generator. It was necessary to open up quite a large section of this wall, in order to get the generator through into its location. I recall that at the time when we estimated this particular job I couldn't figure out just exactly what the coursing was in the brickwork, because it did not conform to the usual dimensions. We found, when we broke through this particular wall, that there was a two-inch air space between the outside course and the rest of the wall. I can't recall exactly how old that particular structure was but it seems to me it was built in the 1870's, with nothing but sand-lime mortar.

As I recall, the wall was approximately 16 ft high. I could see no tie of any kind between the main body of the wall and the outside course. I took a piece of furring and made a complete arc, from the foundation on one side up to the plate and back down to the foundation, without touching a tie of any kind. Still that four-inch wall, 16 ft high, was just as straight and true as it was the day it was built and it had been standing there for some 50 years.

To overcome this leaky building construction, as I previously stated, care should be taken to maintain this vault between the outside four inches of brick and the inside body of the wall.

Construction Costs

The other steps throughout the construction procedure of water-works structures are more or less routine. We haven't much complaint to make, other than that now we are faced with abnormal building costs, and I believe it is going to be quite a considerable length of time before those costs come down much. I don't look for any direct lowering of labor's wages, even though a depression might come along. I think that the fact which stood out in the last depression—the lack of buying power—kept us down perhaps a little longer than would have been the case, if we had maintained the buying power. For this reason, the government will probably spike the wages, as soon as a depression becomes evident.

As far as the materials and manufacturers are concerned, I don't look for much reduction in the direct cost of labor for them, until we get more production per man-hour. With no further wage increases, more efficiency in labor production, and competition cutting into profits, we might possibly see a slight reduction in the cost of the materials, when it becomes a buyer's market, instead of a seller's market, as we have now. Frankly, I don't believe that in the period of the next five years, and perhaps even longer, we are going to see much reduction in the cost of construction of any type of water-works structures.

Scarcity of Materials

One of our big problems today, of course, in the construction of these works is the obtaining of materials. I can remember, back in the days before the war, that when a water-treatment plant would

come up for bids, there would be quite a considerable number of salesmen—quite a number of bidders and sub-contractors—around at the time the bids were opened. Everybody was hungry for his part of the business. Recently, when a bid was opened, we did not even find a cement salesman there, and that is something!

If you happen to be a successful bidder, there is hardly anybody that comes around to offer you an opportunity to purchase his material. You have to call up your friends and plead with them to take some part of the order. They don't know when they are going to be able to deliver it to you, neither will they take the order at any kind of a fixed price. We hope and look for some relief along these lines within the next 12 months, if we don't get another steel strike next month. Anybody who has anything to do with castings is having trouble, because of the shortage of pig iron, and from a contractor's point of view we are still bothered with structural steel. However, our reinforcing steel at the present time isn't too bad, although the lack of detailers in the fabricating plants has delayed the fabrication and detailing of the steel, but the material itself seems to be coming through a little bit better. The structural-steel situation is still bad at the present time. If we get another steel strike in February, I don't know what we are going to do this coming year.

The steel companies are now operating approximately 6 months behind on deliveries and there is some question in our minds, as far as construction contracts are concerned, whether they will be able to better that much in the next 12 months, even though their production goes along without interruption. The engineers and architects have such a backlog of plans and specifications that are either completed or on the boards, nearly completed, that the minute this material begins to be available, a flood of work is going to come out for bids that will still maintain the delay of delivery. However, we are hopeful that we shall be able to get down to a stabilized cost sometime soon, where we do not have to bid on the contingencies, due to inability to obtain firm prices. So far, our experience has been that the owner has not paid any more for a job we complete today, because the contingency items we carry, to cover rising prices, come close to what we have to pay by the time the job is completed.

Conclusion

I have followed this particular line of work in the water-works field, because to me it is one of the most fascinating lines of construction work. This type of work never lacks interesting problems to be solved. We have gone into it to a point where we install most of the equipment. I hope and expect to stay in this particular line of work as long as I am active in the construction industry. I hope that what I have had to say may give you a little bit of an idea of some of the problems that we have and some of the ways that we have found to correct some of the misgivings of previous years. I hope that there is no offense taken, if I have trodden on anybody's toes.

Discussion

Roger W. Esty (Danvers, Mass.). Mr. Munroe, if there was ever a time when I got my money's worth from this New England Water Works Association, I have surely got it this afternoon from the valuable information that you have given to us.

I have had quite a lot of experience this past year on some deep ditching. Sewer trenching is a little bit different from water trenching, in that on water trenching up to 5 ft deep you can battle the water and other things quite easily and successfully but, when you go down 20 ft, that is a different proposition. We did this year on a job just as you suggested about getting some borings.

We had a 20-foot cut. I had never experienced that kind of work before. We didn't know definitely what type of materials was there and we went out and hired a concern and put in some borings. We put them in at every 100 ft—and that follows somewhat what you said, 50 ft back from every one of those center points—with the idea that, if we didn't get enough information at every 100 ft, we would put them in at 50. From that we found, in a distance of some 400 ft where we did the construction, some four feet of a sandy gravel material—no stones over perhaps ¾ of an inch—then we struck a blue clay for probably anywhere from 12 to 15 ft. Sometimes it went the full distance down; sometimes after we went through a brown clay we ran into a blue clay, and in one of the sections the last two or three feet consisted of hard-pan material. Also, from

that work we were quite sure we were not going to strike any great amount of water, if any, right at the bottom.

We started in on the job and went from a shelving job to an open-trench job. We wanted a trench about four feet wide at the base, because we were putting in a 24-in. pipe, and we cut the top 12 or 14 ft wide. We did practically everything in 60 days.

The only place we struck any water of any consequence was near the end of the job. We did have a little water running through the last 300 ft—perhaps 4 or 5 in. came up during the night.

That proved to us the value of going ahead and getting some borings before we started the job. We were so satisfied with what we were finding that we did the whole area, even places where it was only going 8 ft.

Then the thing we were quite interested in was the well points. I have given that a lot of thought, because we are going through an area where last fall, when we had one of the driest falls we have ever had in October or November, when the rainfall was less than an inch, we had ground water about 30 in. from the top of the ground and we had to go 8 or 9 ft deep there. We thought perhaps we would have to go into this well-point system. You frightened me a little bit in that you said it cost $200 a day to operate it. Does that mean that is hired equipment or your own equipment?

JAMES E. MUNROE. We have found that it is a waste of money to purchase well-point equipment. If you purchase well-point equipment, by the time you have used up the rental on that equipment it will be a total loss, anyway. You will not have men around with experience enough to keep it in good condition and repair for the various jobs you will use it on during its life. If you hire the equipment, you are getting new and fresh equipment each time or it has been completely rebuilt. It is poor economy to own one of these systems. The rental on the equipment is a small portion of the cost of operation.

If the job is large enough to warrant the use of 8-inch pumps and if your line is long enough and you are going to get a large volume of water, then the thing to do is to put on a Diesel pump, because you will get by with 35 gal of Diesel oil against 150 gal of gasoline in a day. Your large cost in operating a well-point system is the cost

of your operators. You have to have four men there, working 6 hours a day, 7 days a week, and you have to pay them a sum total of about $250 a week.

ROGER W. ESTY. Well, that was one of the ways we thought we might be able to go through this spot, by unwatering it and going right through with our trenching machine, and with the top soil we have there, we thought we probably could do the job easily.

JAMES E. MUNROE. If you have a free-flowing water in the material, by all means use a well-point system, irrespective of what it costs, because you cannot sheath trenches for the same amount of money. You keep your well-point system strung out ahead of you and you can go right down the street, just as though you were working an absolutely dry street.

As to the cost of a well-point system in regard to the volume of water, there is very little, if any, relationship. I would much rather see a coarse gravel with millions of gallons of water behind it, than I would to be in a tight clay or tight, fine sand, where I could not get the water to the points. If the water won't go to the points, you are not going to get it out of the ground and you are still going to have a bad condition to contend with at the bottom of the trench.

However, a man has to be rather careful, if he has not had too much experience with well points. The four well-point concerns that are in the east have a bunch of super-salesmen and they will often sell you a terrible "bill of goods." You have to take it with a grain of salt and use your own judgment. Many times I have all but practically thrown some of their representatives out of our office because of their crazy ideas. I could stand here for two hours and tell some of the fancy ideas that they come out with that were impractical.

ROGER W. ESTY. I have been watching one in a neighboring city this past summer, where they put in a well-point system right next to a brook—about 20 ft away—and in putting in this underground structure, the men have been running around with no rubbers on. Still there is a stream of water running.

JAMES E. MUNROE. I believe that is in Salem.

ROGER W. ESTY. That is at Peabody.

JAMES E. MUNROE. That was one of the cases where one of the well-point boys came around and said he could dry up that exca-

vation and there was no need of any sheathing at all. He had not looked closely enough, neither had he studied the borings of the ground strata, to see that there was a layer of peat lying on top of the very fine sand and that a 54-inch brick sewer was encased in the peat. There was a cemetery up there on the hill and usually a cemetery is an indication of good loose digging, because when they locate sites for cemeteries, they do not care about digging in hardpan. The water will come down through this material to the top of the fine sand and work its way under the peat. If we hadn't had that steel sheathing in place, the whole street, including the sewer, would have slid right into the hole we had excavated.

The well-point man insisted he would guarantee that he would dry that hole up without any sheathing for so many thousand dollars. He was crazy and I had to argue with him for half a day to get him to forget that idea. His idea was about $15,000 short of what it was going to cost to handle the water. In fact, the material that was at the bottom of the excavation was almost the limit, as far as fine material goes, to dry it out with well-points.

ROGER W. ESTY. Is that your job there?

JAMES E. MUNROE. Yes, sir, although I haven't seen it since the day I went up there to look it over.

ROGER W. ESTY. Another thing you brought out that was very interesting—and we ran into the same thing—was going across a swamp. We stripped the muddy substance off first and then we had a very fine sand and gravel material. We went along quite a little distance, until we came to the first drain, and were going to put in a manhole. There we ran into some really tough digging—that quicksand stuff you spoke about. We were quite baffled and my assistant had been hauling in stone. We thought perhaps the only way to solve it was to keep piling in rock. Finally we got a good, firm foundation in there. We had thought of a sump-pump, drawing the water up through the rocks, and that dried it up. That idea worked out there very nicely.

You spoke about the brickwork. I think our pumping station, which was built back in 1876, was built on the same idea you spoke about—that lime and sand—because we have never had any leakage and the brickwork is just as good today in that building as it was

when it was constructed. As we are considering building another building, some of those facts you just told us about—air spacing, etc.—might be well worth considering. I think all those things you brought out are very fine and I think everybody got something out of your talk.

JAMES E. MUNROE. I might say that, if there are any members of the New England Water Works Association who have any questions along those lines that might come up in the future—whether they are consulting engineers, salesmen, water-works superintendents, committee members or water commissioners—upon receipt of a letter at our office in North Attleborough we would attempt to answer the question to the best of our ability.

ARTHUR L. SHAW (Boston, Mass.). Is there anything else to add?

ALBERT E. CASEY (Stoneham, Mass.). I would like to ask a question. If I understood you correctly, you said the outlook was that the cost of construction would not immediately tumble. I think the public's attitude is that anything in the way of a capital expenditure should be delayed perhaps 5 years, because present-day costs are high and they will be materially reduced 5 years from now. Do I understand you correctly, Mr. Munroe?

JAMES E. MUNROE. That is right. I can't see how it will even come down appreciably in 5 years. I don't think that our labor is going to drop much and that carries right straight through, from the taking of the iron ore from the iron mines right through all of your processing, up to actually erecting it into a structure or any other line of manufacture. The only ways I can see that we are going to have any chance of bringing down our present-day costs are through efficiencies that might be invented and other cuts that might be taken, or through competition that will cut down percentage of profits.

ALBERT E. CASEY. Then you would say that, if something is acutely needed now and you had your choice of perhaps putting it off 5 years, you wouldn't save anything by putting it off?

JAMES E. MUNROE. I don't think you are going to save anything by putting it off. If the facility is to be of some economic value, I am sure that the economic value of having the facility in the period between now and 5 years from now would more than offset any decrease in cost.

The only thing that we are faced with is the lack of materials right at the present time and we have to spread out the work to conform to the materials that are available. The minute that our steel companies get rolling, there is going to be an immediate flooding of the market, for a month or two, with a lot of new projects. Right now your structural steel is in the third-quarter rolling and you cannot get any information from the mills as to what part of the third quarter it is. It is likely to be the 30th of September before you get it out of the mill.

ARTHUR L. SHAW. Mr. Munroe, Mr. Esty has expressed an interest in vaulted brickwork and very likely some others, who didn't know of that method of waterproofing a brick wall, have been wondering how you get the water out of the foot of the wall. You didn't cover that in describing it.

JAMES E. MUNROE. Usually what we do is this: in the second or third course above the foundation, due to the fact that the bricklayers allow a certain amount of mortar to drop down and fill up a course or two in that vault, we place twenty-penny cut spike in the brick mortar, when we get up to the third course, work it around and let it stay there, until the brickwork is finished, and any accumulation of water that may get into the vault will run out through that hole.

I have seen some cases where little, round brass pipes were specified but I don't think that is necessary. The chances are they wouldn't be put in there, unless the boy who had the book happened to see it and made a particular point of checking to see whether they were in there or not.

ARTHUR L. SHAW. I think that was your idea on the Lawrence plant, putting in those little copper tubes.

JAMES E. MUNROE. I brought that point up to the engineers regarding the vaulted wall. The specifications required that the brick walls should be slushed solidly and that the contractor should guarantee that the walls would be water-tight. Inasmuch as there was a direct conflict there, I had to go to the engineers' office to find out whether they wanted a water-tight wall or a slushed wall. They were primarily interested in a water-tight wall and, therefore, it was not slushed. As far as I know, those walls have not leaked to any great

extent. I believe that, if they did, it was only dampness on some of the headers on the inside face, that went through the wall, but as a whole I think the structure was quite tight.

ARTHUR L. SHAW. That vaulting is a very excellent way to provide for water that strikes through the course of brick. Are there any other questions or discussions?

A MEMBER. If you get one of these walls that leaks, is there any way that you can stop a leak?

JAMES E. MUNROE. Well, there is another part of the construction industry that something should be done about, and that is the waterproofing industry as a whole. The leakage can be stopped by the use of a colorless waterproofing. There are several of them on the market. Some are better than others. If the material is properly applied according to the manufacturer's directions, the leakage can be stopped. However, most of these waterproofing contractors agree to do something and don't do it, and the job is only half done, or they dilute the material, whitewash it on there, and get in and get out, because nobody happens to be watching them, with the result that there is little or no benefit from the expenditure of the money, but the leakage can be stopped, if the work is properly done.

The only other way is raking the joints out and repointing the brickwork, and that involves a terrific cost per square foot. It is an impractical thing to do from the point of view of cost.

ARTHUR L. SHAW. Are there any other questions? If not, I declare the meeting adjourned, and thank you very much, Mr. Munroe.

THE CHARACTER OF QUABBIN RESERVOIR WATER DURING RESERVOIR FILLING

BY CLARENCE H. REED*

[*Read at Chemists' Round Table, February 27, 1947.*]

QUABBIN Reservoir is 18 miles long and covers an area of 38.6 sq mi; it has 177 miles of shore line, including the shore lines of over 100 steep-sloped islands. The elevation of the spillway is 530 ft above sea level; the maximum depth is 148 ft and the average depth about 51 ft; the capacity is 415 billion gallons.

The bottom of Quabbin Reservoir was cleared to an elevation above the shore line but was not stripped of the top soil.

It required nearly seven years to fill the reservoir—from August, 1939, to the spring of 1946. During this time a continuous flow of 20 mgd was required to be released to the old river bed for mills downstream and about 96 billion gallons were drawn to the Wachusett Reservoir, for consumption in the Metropolitan District. During the filling, approximately 89 billion gallons were taken from the Ware River to Quabbin Reservoir.

In Figure 1 is shown the time of filling Quabbin Reservoir, together with the volume of water in storage as filling progressed. Table 1 shows the elevation of the reservoir and the increase in storage, by years.

The variation in net increase in storage from year to year was due either to the yield or amount drawn for consumption, or both.

The main feeders to Quabbin Reservoir are the three branches of the Swift River and what might be called the flood flow from the Ware River. Listed in Table 2 are the average results of analyses of these rivers for the 7-year period, 1940 to 1946, inclusive.

From the three branches of the Swift River, samples were collected twice a month during 1940 and 1941, and once a month thereafter. The results of analysis of the Ware River are from samples

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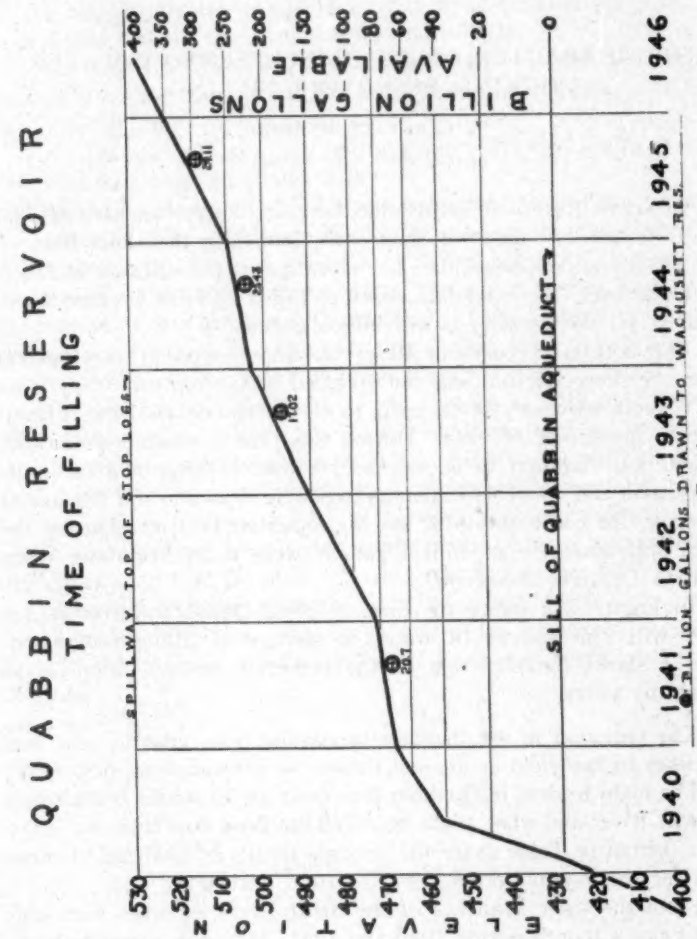
QUABBIN RESERVOIR
TIME OF FILLING
SPILLWAY TOP OF STOP LOGS
SILL OF QUABBIN AQUEDUCT
BILLION GALLONS AVAILABLE
400
350
300
250
200
150
100
80
60
40
20
0
530
520
510
500
490
480
470
460
450
440
430
420
410
400
FEET ABOVE MEAN SEA LEVEL
1940
1941
1942
1943
1944
1945
1946
297
1032
2054
2911
BILLION GALLONS DRAWN TO WACHUSETT RES.

FIG. 1.

TABLE 1.—ELEVATION AND INCREASE IN STORAGE, BY YEARS

Date	Elevation	Increase	
		Feet	Billion gallons
Started Aug. 5, 1939	392.51		
1939	422.00	29.5	6.1
1940	462.20	49.2	72.2
1941	472.15	2.95	10.0
1942	489.12	17.0	63.7
1943	500.72	11.5	59.3
1944	507.29	6.5	37.0
1945	520.78	13.5	94.0
1946	528.60	7.2	58.5

TABLE 2.—AVERAGE RESULTS OF ANALYSES OF SWIFT AND WARE RIVERS FOR SEVEN-YEAR PERIOD, 1940-1946, INCLUSIVE

	Swift River				
	West Branch	Middle Branch	East Branch	Average of West, Middle and East Branches	Ware River
Turbidity—silica standard	1.6	2.6	3.1	2.4	2.1
Color—platinum standard	21	39	49	36	52
Oxygen consumed—ppm	4.6	6.7	8.2	6.5	8.2
Chlorine—ppm	1.2	1.2	1.1	1.2	1.2
Alkalinity—ppm	6.0	7.3	7.2	6.8	4.8
Hardness, soap method—ppm	13	15	14	14	12.2
H-ion concentration—pH value	6.5	6.5	6.5	6.5	6.3
Iron—ppm	0.10	0.25	0.60	0.32	0.27
Residue on evaporation (ppm):					
Total	36	40	45	41	41
Suspended	3.1	5.0	6.8	5.0	4.3
Volatile matter (ppm):					
Total	16	20	23	20	20
Suspended	2.2	3.4	4.5	3.4	3.1
Dissolved oxygen—ppm	11.1	9.1	9.1	9.8	11.2
Dissolved oxygen—% saturation	93	89	84	89	90
Oxygen demand (5 days, 20° C.)—ppm	0.52	1.1	1.3	1.0	0.8
Manganese	Sl. trace	Sl. trace	None	Sl. trace	Sl. trace
Bacteria per cc, 37.5° C.	103	100	115		97
Bact. coli in 100 cc— Phelps index	77	127	185		80

collected only during the periods when water was being taken from the Ware River to Quabbin Reservoir and represent the river when the flow was above average or the river was in a flooded condition.

For analytical data, depth sampling points in Quabbin Reservoir, known as Stations 202 and 206, were selected. Station 202 is located approximately ¼ of a mile above Winsor Dam. The 20 mgd of water that are required to be released from the reservoir, as mentioned before, were drawn from the bottom for a few months after the reservoir started to fill, or until the reservoir was about 48 ft deep at the dam and 33 ft deep at Station 202; thereafter, the water was released from the outlet works at an elevation above sea level of 428 ft and at 467 ft when it reached that level. The elevation of the bottom at Station 202 is 397 ft above sea level.

Station 206 is located near the extreme west end of Quabbin Aqueduct and represents the water that is drawn from the reservoir for consumption. The elevation of the bottom at Station 206 is 422 ft above sea level.

Depth samples from these stations were collected twice a month. All samples called "bottom" in this paper were collected from 5 to 10 ft above the bottom of the reservoir.

Temperature

To show seasonal variations in temperature at various depths, four seasonal periods are taken in 1946, the results being presented in Table 3 and in Figure 2:

TABLE 3.—TEMPERATURES (°C.) IN QUABBIN RESERVOIR, 1946

Depth (ft)	Jan. 30*	June 5†	Aug. 26†	Nov. 27†
Surface	1.0	16.0	22.0	11.0
10	1.0	14.5	21.0	11.0
30	1.0	13.5	21.0	11.0
50	1.5	11.5	15.0	11.0
70	2.0	11.0	13.0	11.0
90	2.0	10.0	13.0	11.0
110	2.0	9.0	12.0	11.0
122	2.0	9.0	11.0	11.0
132	2.0	8.5	11.0	11.0

*Elevation of surface, 519 ft.
†Elevation of surface, 529 ft.

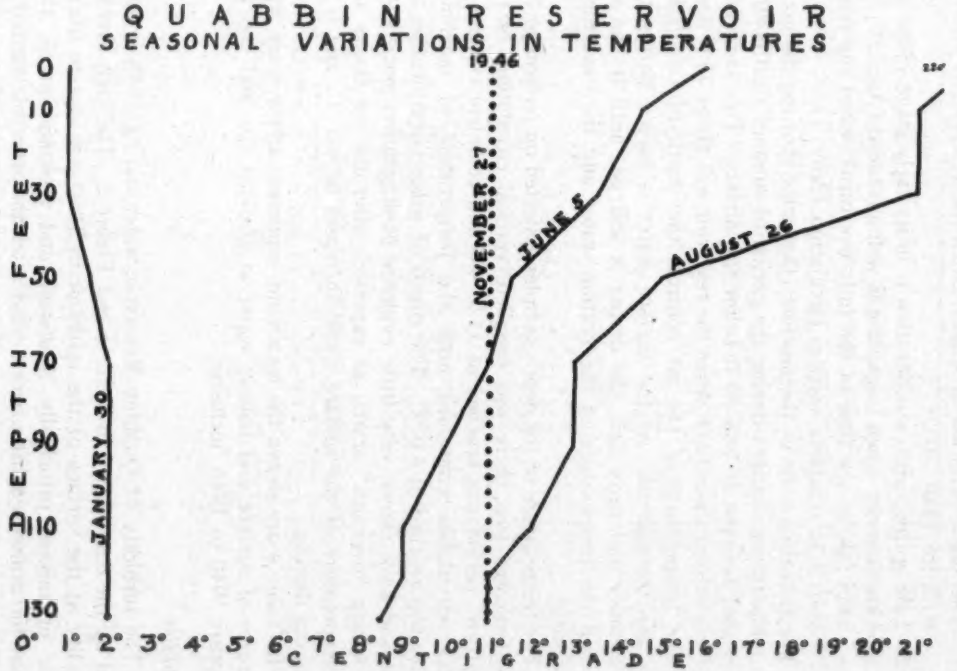
QUABBIN RESERVOIR
SEASONAL VARIATIONS IN TEMPERATURES
DEPTH IN FEET
0
10
30
50
70
90
110
130
JANUARY 30
NOVEMBER 27
1946
JUNE 5
AUGUST 26
0° 1° 2° 3° 4° 5° 6° 7° 8° 9° 10° 11° 12° 13° 14° 15° 16° 17° 18° 19° 20° 21°
C E N T I G R A D E

FIG. 2.

1. The slight variation in winter, when the reservoir is covered with ice (Jan. 30).
2. Late spring, when stratification is in its early stage (June 5).
3. Late summer, when stagnation is well advanced (Aug. 26).
4. Late fall, at the time of the fall "overturn", when the reservoir is in complete vertical circulation (Nov. 27).

The transition zone or thermocline (that zone showing the maximum temperature change) during the period of summer stratification was located between 30 and 50 ft below the surface. This same condition has existed since 1941, when the reservoir was about 75 ft deep.

The temperature of the air controls the depth of circulation above the thermocline. As the surface water is cooled during the late summer and early fall, the deeper it will go, until it has approached the temperature of the bottom water and the "overturn" is complete.

The temperature of the depth samples collected on November 27, 1946, indicated that there was complete vertical circulation, or the "overturn" period was reached at 11.0° C., and complete vertical or thermal circulation continued until the temperature of maximum density was reached at 4.0° C. The effects of winter stratification are slight and have shown very little evidence of stagnation since 1942. The spring "overturn" starts, as expected, after the ice thaws and the temperature of the surface water increases to 4.0° C. and is at maximum density.

In Table 4 are shown the maximum, minimum and average temperatures of surface and bottom water at Stations 202 and 206 for the years 1940 to 1946, inclusive.

Turbidity

The turbidity of Quabbin Reservoir water during 1940 is indicated by the data shown in Table 5 and Figure 3. The high average turbidity at the surface of the reservoir in 1940 was due to microscopic organisms, principally *Anabaena* and *Aphanizomenon,* that existed for a month a little over a year after the reservoir started to fill. The counts of *Anabaena* ranged from 2,000 to 2,000,000 standard areal units per ml.

During this period there was a slight increase in dissolved oxygen

TABLE 4.—SURFACE AND BOTTOM TEMPERATURES (°C.) AT STATIONS 202 AND 206

Year	Depth of reservoir in Dec. (ft)	Surface Max.	Surface Min.	Surface Aver.	Bottom Max.	Bottom Min.	Bottom Aver.
			Station 202				
1940	72	25.0	7.0*	13.1	13.5	6.0*	10.4
1941	75	25.0	0.5	15.0	14.0	1.5	11.1
1942	92	24.0	1.0	13.8	12.0	2.0	8.6
1943	103	25.0	1.0	14.3	12.0	3.5	8.3
1944	110	26.5	2.0	15.5	12.0	3.0	8.8
1945	123	25.0	1.5	14.3	13.0	3.0	8.9
1946	132	23.5	1.0	14.1	12.0	2.0	8.9
			Station 206				
1940	47	25.5	7.0*	17.4	15.0	7.0*	13.0
1941	50	24.5	1.5	16.9	15.0	3.0	14.6
1942	67	24.5	2.0	13.4	16.5	2.5	11.0
1943	78	25.0	1.0	14.1	13.0	1.0	8.9
1944	85	25.0	2.0	15.6	14.5	3.0	9.9
1945	98	25.0	0.5	15.7	14.5	3.0	10.2
1946	107	23.0	0.5	13.4	12.0	2.0	9.8

*Samples were not collected until April 29.

TABLE 5.—TURBIDITY (ppm) IN 1940

Depth (ft)	Aug. 10	Sept. 4	Sept. 16	Oct. 2
Surface	7	20	13	7
6	8	20	10	7
16	4	15	11	6
26	5	6	8	7
36	5	7	6	7
46	6	9	5	6
56	6	6	5	13
66	11	5	10	10

and a definite increase in pH from 16 to 26 feet below the surface. This condition was general at all sampling stations throughout the reservoir; a similar condition has not been observed since. There was a marked decrease in turbidity during the next two years at both

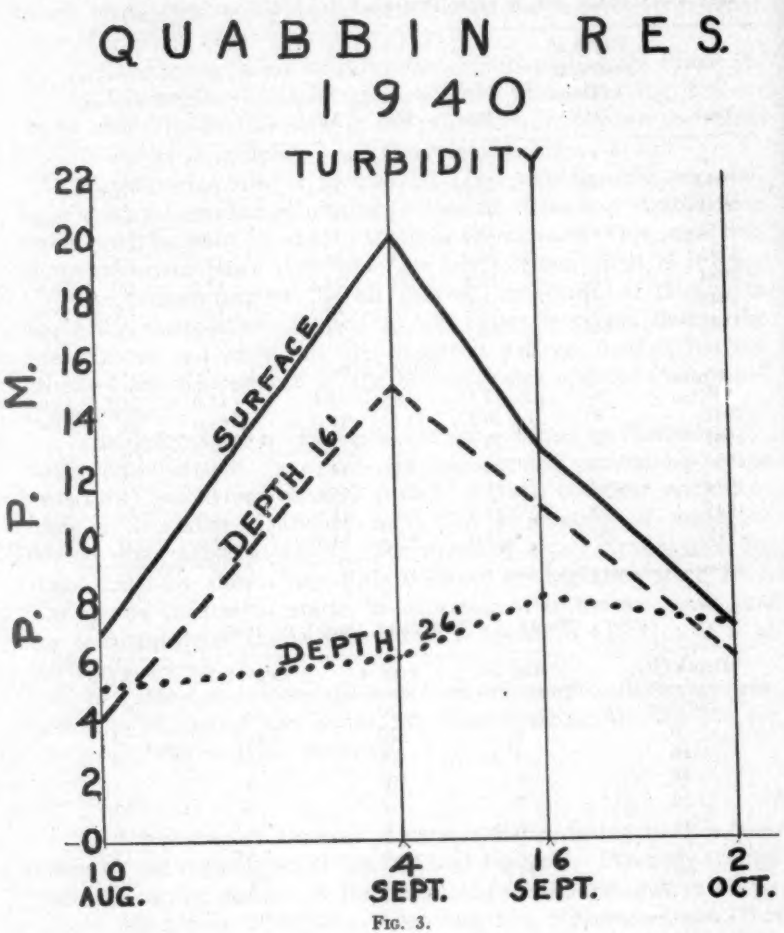
QUABBIN RES.
1940
TURBIDITY
P. P. M.
22
20
18
16
14
12
10
8
6
4
2
0
SURFACE
DEPTH 16'
DEPTH 26'
10 AUG.
4 SEPT.
16 SEPT.
2 OCT.

FIG. 3.

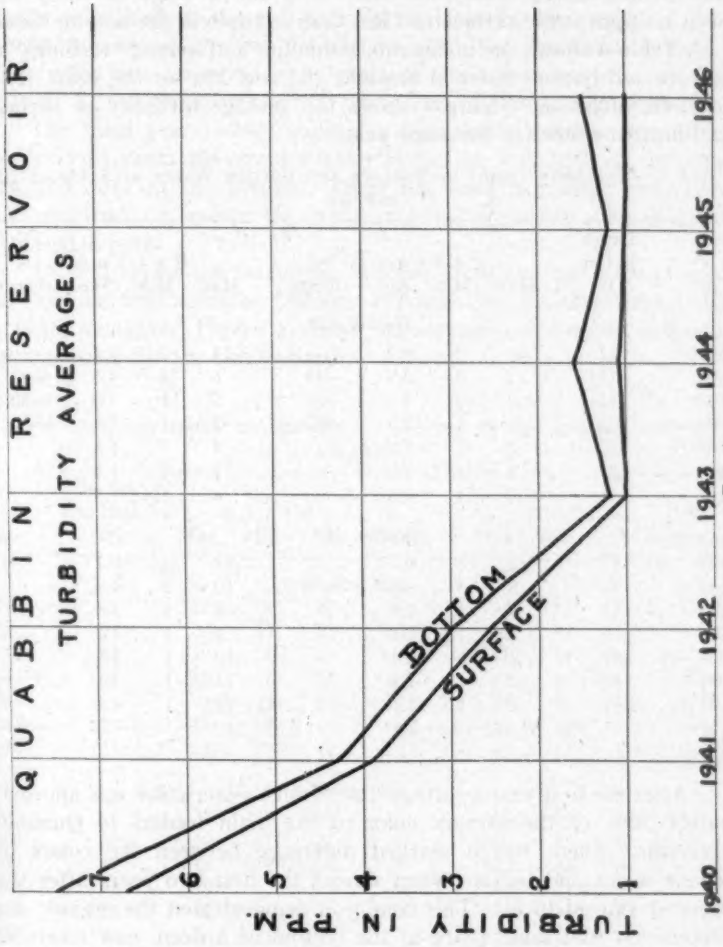
QUABBIN RESERVOIR
TURBIDITY AVERAGES
TURBIDITY IN PPM.
7
6
5
4
3
2
1
1940
1941
1942
1943
1944
1945
1946
BOTTOM
SURFACE

FIG. 4.

the surface and bottom, and since 1943 the average turbidity has been 1.1 ppm at the surface and less than 2.0 ppm in the bottom water.

Table 6 shows the maximum, minimum and average turbidity of surface and bottom water at Stations 202 and 206 for the years 1940 to 1946, inclusive. Figure 4 shows the average turbidity of surface and bottom water for the same years.

TABLE 6.—TURBIDITY (ppm) OF SURFACE AND BOTTOM WATER AT STATIONS 202 AND 206

Year	Depth in Dec. (ft)	Surface Max.	Surface Min.	Surface Ave.	Surface % change	Bottom Max.	Bottom Min.	Bottom Ave.	Bottom % change
				Station 202					
1940	72	20	3	6.7		14	2	7.5	
1941	75	5	3	3.9	—42	6	3	4.3	—43
1942	92	4	2	2.6	—33	7	1	3.0	—30
1943	103	1	1	1.0	—62	2	1	1.2	—60
1944	110	2	1	1.1		4	1	1.6	
1945	123	2	1	1.0		2	1	1.2	
1946	132	2	1	1.0		4	1	1.5	
				Station 206					
1940	47	12	3	6.7		12	3	6.7	
1941	50	6	3	4.8		10	3	4.8	
1942	67	5	1	2.8		8	1	3.8	
1943	78	2	1	1.2		9	1	2.2	
1944	85	2	1	1.4		10	1	2.6	
1945	98	2	1	1.1		14	1	2.5	
1946	107	2	1	1.1		18	1	4.3	

Color

After the first year's storage, the surface-water color was approximately 50% of the average color of the main feeders to Quabbin Reservoir. There was a marked difference between the colors of surface water and bottom water during the first two years after the reservoir started to fill. This condition demonstrated the organic decomposition that takes place at the bottom of a deep, new reservoir during the summer stratification period, causing increased carbon dioxide and resulting in the solution of the bottom deposit of organic matter along with any mineral salts of iron and manganese.

The average color of the bottom water during the first two years was twice as high as the surface color and, during the period of advanced summer stagnation the second year, the maximum bottom-water color was seven times as great as the maximum color at the surface. At this time the reservoir was about 75 ft deep at Station 202.

The third year (1942) showed a remarkable improvement over the previous year; the average color reduction was 26% at the surface and 56% at the bottom. There has been a gradual reduction each year to date, except for the year 1944, when the average color increased 1 ppm.

Table 7 shows the maximum, minimum and average color of surface and bottom water at Stations 202 and 206 for the years 1940 to 1946, inclusive. Figure 5 shows the average color of surface and bottom water for the same years.

TABLE 7.—COLOR (ppm) OF SURFACE AND BOTTOM WATER AT STATIONS 202 AND 206

Year	Depth in Dec. (ft)	Surface Max.	Surface Min.	Surface Ave.	Surface % change	Bottom Max.	Bottom Min.	Bottom Ave.	Bottom % change
				Station 202					
1940	72	25	18	20		95	22	41	
1941	75	27	17	23	+15	195	18	46	+12
1942	92	18	14	17	—26	41	14	20	—56
1943	103	18	14	16	— 6	22	14	17	—15
1944	110	18	16	17	+ 6	25	16	18	+ 6
1945	123	18	12	15	—12	22	12	16	—11
1946	132	14	12	13	—13	20	12	15	— 6
				Station 206					
1940	47	33	18	22		60	22	31	
1941	50	40	15	22		130	18	43	
1942	67	21	17	16		190	16	43	
1943	78	19	17	16		44	17	21	
1944	85	18	16	17		47	16	21	
1945	98	18	12	15		33	13	18	
1946	107	14	12	13		65	13	19	

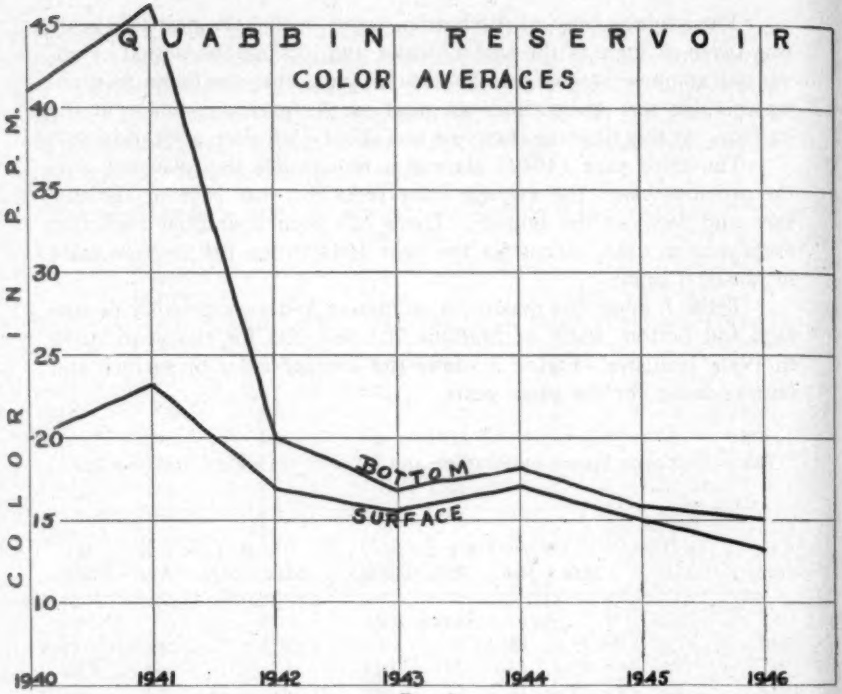

FIG. 5.

Hydrogen-ion Concentration

The hydrogen-ion concentration (pH) averaged about 6.7 at the surface and 6.4 at the bottom. As is shown in Table 8, there has been no marked change in the average pH since the reservoir started to fill.

Iron

The average iron results showed a slight increase in 1941, the second year after the reservoir started to fill, and a decided decrease in 1942 and 1943, the third and fourth years, both at the surface and at the bottom. Since 1943 the content of iron at the surface has shown a gradual decrease, while the average content of iron in the bottom water has remained approximately the same.

TABLE 8.—HYDROGEN-ION CONCENTRATION (pH) OF SURFACE AND BOTTOM WATER AT STATIONS 202 AND 206

Year	Depth in Dec. (ft)	Surface Max.	Surface Min.	Surface Aver.	Bottom Max.	Bottom Min.	Bottom Aver.
		Station 202					
1940	72	7.0	6.3	6.7	6.7	6.4	6.5
1941	75	7.4	6.5	6.8	6.7	6.2	6.5
1942	92	7.0	6.6	6.7	6.8	6.0	6.4
1943	103	6.9	6.5	6.7	6.6	6.2	6.4
1944	110	6.9	6.5	6.7	6.7	6.1	6.4
1945	123	6.9	6.5	6.7	6.6	6.3	6.5
1946	132	6.9	6.5	6.7	6.7	6.2	6.4
		Station 206					
1940	47	7.1	6.5	6.7	7.0	6.4	6.6
1941	50	7.3	6.6	6.8	6.9	6.4	6.6
1942	67	7.2	6.3	6.8	6.7	6.2	6.5
1943	78	6.9	6.6	6.7	6.8	6.1	6.3
1944	85	7.1	6.5	6.8	6.7	6.1	6.4
1945	98	6.9	6.6	6.7	6.6	6.1	6.3
1946	107	6.9	6.5	6.7	6.7	6.1	6.3

Table 9 shows that the iron content was low for bottom water in a large, deep reservoir.

Figure 6 presents the maximum and average iron content of surface and bottom water for the years 1940 to 1946, inclusive.

Manganese

The manganese, like the color and iron, increased at the surface and through the lower region to the bottom of the reservoir the second year after the reservoir started to fill, with a marked decrease the third year (1942). After the first year, in 1940, manganese extended from the bottom upward to within 18 ft of the surface. The maximum content at the bottom was 0.5 ppm, with a gradual decrease to a trace at 18 ft below the surface. This was during August, when summer stagnation was well advanced. After the fall "overturn" in October, it did not appear again until March, 1941. At this time it extended from the bottom upward, half-way to the surface, and again

TABLE 9.—IRON (ppm) IN SURFACE AND BOTTOM WATER AT STATIONS 202 AND 206

Year	Depth in Dec. (ft)	Surface Max.	Surface Min.	Surface Ave.	Surface % change	Bottom Max.	Bottom Min.	Bottom Ave.	Bottom % change
				Station 202					
1940	72	0.55	0.10	0.24		4.8	0.15	1.4	
1941	75	0.75	0.05	0.25	+ 4	5.0	0.15	1.5	+ 7
1942	92	0.40	0.10	0.16	—36	1.8	0.10	0.46	—69
1943	103	0.20	0.05	0.13	—19	0.8	0.10	0.22	—52
1944	110	0.20	0.10	0.12	— 8	0.65	0.10	0.24	+ 9
1945	123	0.15	0.05	0.09	—25	0.60	0.10	0.20	—17
1946	132	0.15	0.05	0.07	—22	0.60	0.10	0.21	+ 5
				Station 206					
1940	47	0.30	0.05	0.16		2.0	0.10	0.58	
1941	50	0.40	0.10	0.25		4.5	0.20	1.20	
1942	67	0.30	0.10	0.18		5.5	0.10	1.20	
1943	78	0.25	0.10	0.15		1.5	0.10	0.55	
1944	85	0.20	0.05	0.12		3.2	0.10	0.62	
1945	98	0.15	0.05	0.09		1.8	0.10	0.33	
1946	107	0.15	0.05	0.09		4.5	0.50	0.62	

subsided to the bottom during the spring "overturn" in April. The winter appearance of manganese indicated winter stagnation. During the summer stratification period of 1941, for nearly five months manganese was present throughout the reservoir, with a maximum content of 1.6 ppm at the bottom and 0.14 at the surface. Sedimentation again took place during the fall "overturn". In 1942, the third year, manganese was carried to the surface only once, in the month of November during the fall "overturn". In 1943 manganese extended from the bottom upward to within 30 ft of the surface. Since 1944 manganese has not been present above the thermocline, or within 50 ft of the surface.

From the analytical results for the first three years, it was interesting to note the sudden reduction of iron and complete disappearance of manganese after the fall "overturn", when they were brought in contact with oxygen and the salts were oxidized to ferric and manganic hydroxides. Complete flocculation and sedimentation of the manganese usually took place within two weeks after the "overturn".

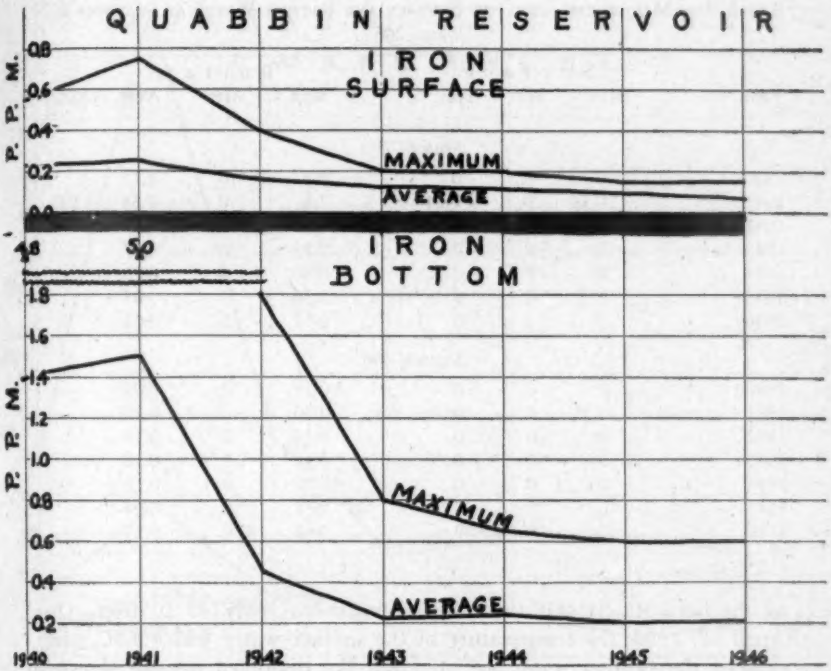

FIG. 6.

Table 10 shows the maximum, minimum and average manganese content of surface and bottom water at Stations 202 and 206 for the years 1940 to 1946, inclusive. Figure 7 shows the maximum and average manganese content of surface and bottom water for the same years.

Dissolved Oxygen

During the first spring after the reservoir started to fill, the dissolved-oxygen results, as well as the appearance of manganese, indicated winter stagnation. On March 20 the surface temperature was 2.0° C. and the bottom temperature was 3.5° C. The dissolved oxygen showed 85% saturation at the surface and 40% saturation

TABLE 10.—MANGANESE (ppm) IN SURFACE AND BOTTOM WATER AT STATIONS 202 AND 206

Year	Surface			Bottom			% change
	Max.	Min.	Aver.	Max.	Min.	Aver.	
			Station 202				
1940	0	0	0	0.5	0	0.13	
1941	0.14	0	0.04	1.6	0	0.38	+190
1942	0.10	0	T	0.55	0	0.14	— 63
1943	0	0	0	0.25	0	0.07	
1944	0	0	0	0.30	0	0.11	
1945	0	0	0	0.30	0	0.05	
1946	0	0	0	0.25	0	0.08	
			Station 206				
1940	0	0	0	0.25	0	0.05	
1941	0.14	0	0.01	0.90	0	0.20	
1942	0	0	0	0.78	0	0.12	
1943	0	0	0	0.38	0	0.10	
1944	0	0	0	0.80	0	0.16	
1945	0	0	0	0.28	0	0.04	
1946	0	0	0	0.65	0	0.14	

T = Trace

at the bottom. At this time the reservoir was only 45 ft deep. On April 15, 1940, the temperature of the surface water was 4.5° C. and that of the bottom water 3.5° C., and the dissolved oxygen showed 79% saturation at the surface and 70% at the bottom. During the summer of 1940, when stagnation was well advanced, dissolved oxygen was depleted in the lower regions of the reservoir for two months, from early August to the first week in October, as shown in Table 11 and Figure 8. This depletion extended upward into the transition zone, to within 30 ft of the surface. At this time the reservoir was 70 ft deep.

It is interesting to note the penetration of dissolved oxygen to greater depths, as the early fall air chilled the surface water and started thermal circulation.

The second year, 1941, when the depth at Station 202 was about 75 ft, the dissolved oxygen did not drop below 5% saturation and, during the months of August and September, it varied from 5 to 10%

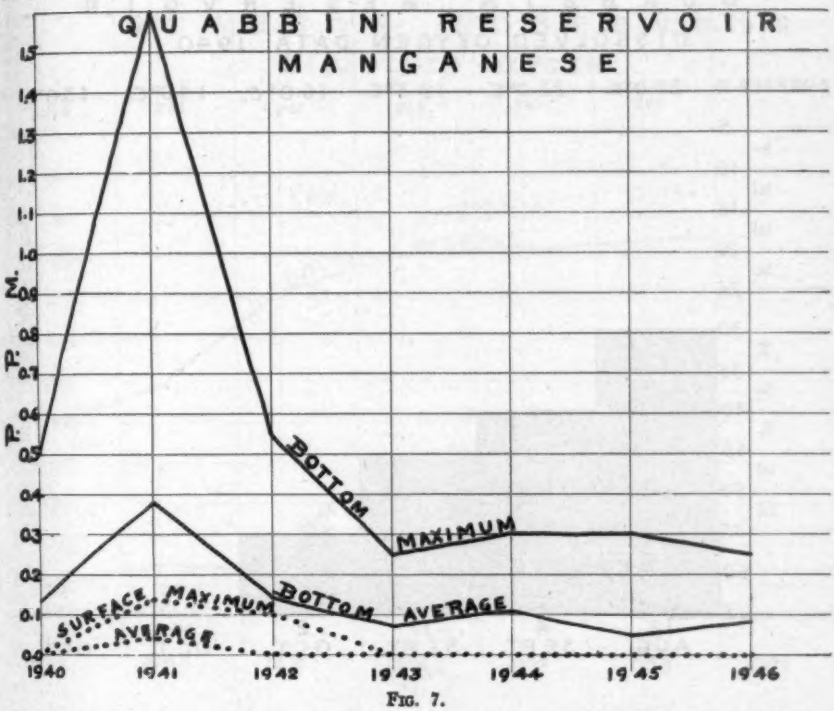

FIG. 7.

TABLE 11.—DISSOLVED OXYGEN IN SURFACE AND BOTTOM WATERS, 1940

Date, 1940	Temperature (°C.) Surface	Temperature (°C.) Bottom	Dissolved oxygen (% sat.) Surface	Dissolved oxygen (% sat.) Bottom	D.O. depleted from bottom to within — feet of surface
Aug. 12	25.0	12.0	99	00	30
Sept. 4	22.0	12.0	106	00	40
" 17	18.5	11.0	81	00	45
Oct. 2	16.0	11.5	70	T	55
" 14	14.0	11.5	66	6.8	65
" 21	13.0	12.0	66	70	
" 29	11.0	10.5	76	76	

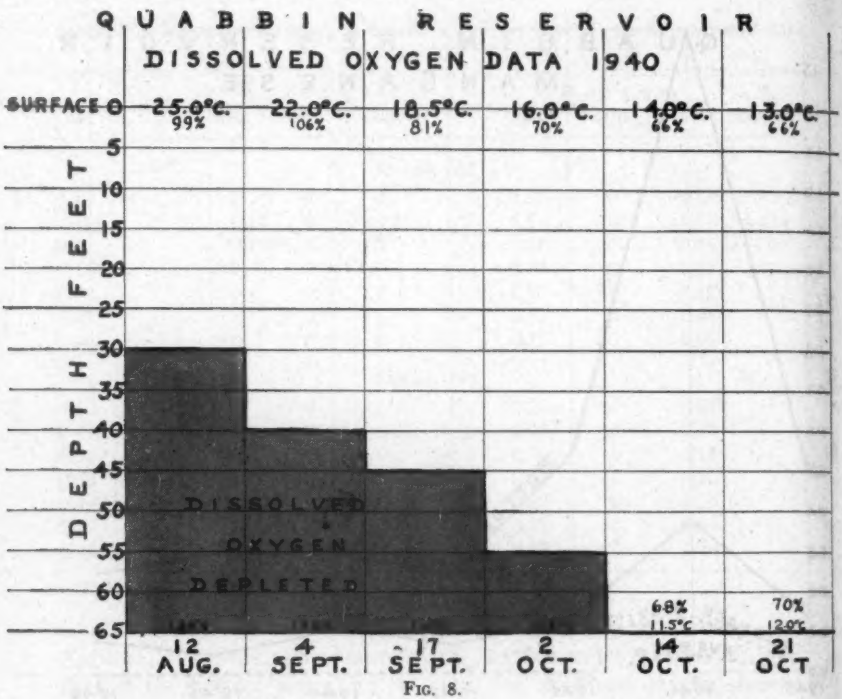

FIG. 8.

saturation. The third year, 1942, when the depth at Station 202 was approximately 90 ft, the minimum dissolved oxygen recorded during advanced stagnation showed 11% saturation, except for one sample collected October 22, just previous to the fall "overturn", which showed only 4% saturation. In 1943, there was no sample collected for dissolved-oxygen determination at Station 202 that showed less than 29% saturation and the average increase for the year was 12.5% over the previous year. Since 1943 the bottom samples have shown a gradual increase in dissolved oxygen, as indicated in Figure 9.

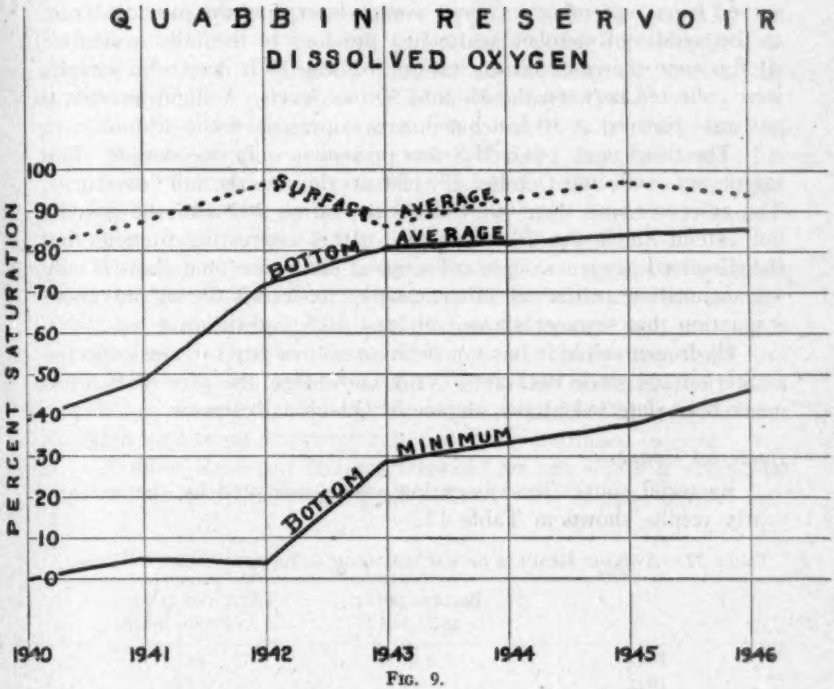

FIG. 9.

Hydrogen Sulphide

During excessive organic decomposition, anaerobic bacterial activities in the lower regions of new, large, deep reservoirs result in the reduction of sulphates and liberation of hydrogen sulphide.

Hydrogen sulphide was noticed in samples collected from the lower regions of Quabbin Reservoir during some of the months of advanced summer stagnation. A little over a year after the reservoir started to fill, H_2S was present from the first week in September to the middle of October, 1940, when the fall "overturn" occurred. At this time the reservoir was about 70 ft deep at Station 202 and the gas extended upward from the bottom to within 36 ft of the surface. Two years after the reservoir started to fill—in 1941—H_2S was ob-

served from September 8 (about a week later than the previous year) to the middle of October again, just previous to the fall "overturn". At this time the reservoir at Station 202 was 78 ft deep. No samples were collected between the 30- and 50-foot levels. A slight amount of gas was observed at 50 feet but none was present at the 30-foot level.

The third year, 1942, H_2S was present in only one sample. This sample was collected October 22, just previous to the fall "overturn". The reservoir was then 90 ft deep at Station 202 and the gas did not extend above the 70-foot level. (It is interesting to note that the dissolved-oxygen sample collected at the same time showed only 4% saturation, while all other samples collected during advanced stagnation that summer showed at least 10% saturation.)

Hydrogen sulphide has not been present in any samples collected at Station 202 since 1942 and, to my knowledge, the odor of H_2S has never been detected at the surface of Quabbin Reservoir.

Bacterial Counts

Bacterial counts have been low, as is indicated by the average yearly results shown in Table 12.

TABLE 12.—AVERAGE RESULTS OF BACTERIOLOGICAL EXAMINATIONS, BY YEARS

	Bacteria per cc at 37.5° C.	Bact. coli in 100 cc (Phelps index)
1941	150	20
1942	13	2.0
1943	31	1.1
1944	55	2.0
1945	16	0.75
1946	75	2.0

Summary

The color of Quabbin Reservoir increased slightly the second year but decreased rapidly the third and fourth years, 1942 and 1943. Since then there has been a gradual decrease, until at present the color is 12 ppm. The color should continue to decrease.

The mineral salts, such as iron and manganese, in line with the color, increased the second year but there was a vast improvement the third and fourth years (1942 and 1943). The contents of both

iron and manganese have remained approximately the same in the bottom water since 1943, while at the surface the iron content continues to decrease. Manganese has not appeared within 30 ft of the surface since 1943 nor above the 50-ft level since 1944.

The dissolved-oxygen content fell to zero, or nearly so, in the lower regions of the reservoir, during summer and fall stagnation periods the first three years, but has not been below 29% saturation since 1942. The average at the bottom has been above 80% saturation at Station 202 and 60% at Station 206 since 1943.

Hydrogen sulphide was present in the lower regions of Quabbin Reservoir the first three years, during the summer and fall seasons. There has been no hydrogen sulphide present since 1942.

In general, Quabbin Reservoir improved rapidly the second and third years—in 1941 and 1942. After 3½ years of filling, the results of analyses showed that a relatively stable condition had been reached, although significant improvement in quality continues to date.

Quabbin Reservoir has now reached an age where it should be governed more by its own deposits than by the original bottom soil.

WATER RATES

BY JOHN B. KENNEDY*

[*Read at Superintendents' Round Table, February 27, 1947.*]

Costs of delivering water to consumers vary widely in different communities and are influenced by many conditions, such as availability of water supply, elevation of source, geological conditions, topography, etc. It is easy to see that it is a decided advantage for a community to be situated in a valley, with a beautiful, natural water supply located high on a nearby hill, from which the water flows by gravity into a distribution system. It is just as easy to see the disadvantage for a community situated on a high plateau, which takes its water from a river below and which must first purify the water and then pump it up to the distribution system.

There are also many communities which must maintain large watersheds, reservoirs, well fields and various types of purification plants. Then, too, there are the communities which have no water supply of their own and must buy their water from outside sources at various prices. There is also a difference in the cost of distribution between large and homogeneous centers of population and sparsely settled communities with long extensions. The difference in the method of financing publicly owned and privately owned water utilities is also a factor.

From the foregoing one can easily see that it would be next to impossible to devise a hard-and-fast rule, by which water rates may be determined. In some sections water rates are set by commissions. In the case of most of us, however, we shall probably be called upon at some time or other to collaborate with the rate-setting authority of our community in the settling of a new water rate or in revising our present rate. I shall not attempt any discussion of flat rates or any outmoded methods of assessing water charges, because I believe we are all interested in modern practices. Of course, we must all realize that we all operate under different laws, rates, regulations,

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plans, local ordinances, boards, commissions, mayors, city managers, etc., that all of our conditions and circumstances vary, and that generally we must "cut the suit to fit the cloth". However, there are a few general ideals which we should all strive to attain.

To begin with, a water-supply system must be designed and constructed in such a manner that it will meet all demands which the community it serves may put upon it. These considerations finally and effectually determine the cost of the system and its operating expenses. Therefore, inasmuch as the water itself costs very little or nothing, it appears that in determining water rates the charges in all except isolated cases are almost exclusively for services or "readiness to serve", rather than for the water itself.

Determination of rates must be expert, just and impartial. Advice on rate-setting is usually requested from the superintendent, because of his expert, first-hand knowledge of the system. Unreasonably low rates endanger the interests of the water takers, the investors and the administrators of the water system. Inadequate rates always eventually result in poor service and in a run-down system. Then both the water takers and the utility suffer.

There should be no free water service. All water service must be paid for by someone. If any system does not charge for all water service, then it becomes obvious that its rates cannot be fair and equitable, because its paying customers are paying for services which they did not receive. Charges for private fire services are hard to determine. Sprinkler services and private hydrants require a lot of "readiness to serve". Even if the water is metered, the meter rate does not usually fully represent the cost of service and a service charge should be worked out.

Because it is a complete subject in itself and we may have a future Round Table discussion on it, I do not want to discuss "fire-prevention revenue" here. However, I do want to point out that in determining our water rates we should take into account this item. Use one of the approved methods of determining it and include in your anticipated revenue an equitable charge for "fire prevention" or "hydrant rental", whichever you prefer to call it.

The use of meters is generally accepted as the most satisfactory and best method upon which to base water charges. In figuring meter

rates, the fairest and most scientific method is to provide a minimum or service charge, to cover the installation, fixed charges, maintenance and customer costs, such as meter reading, billing, etc. Even though the "service charge" is accepted by the courts and approved by the U. S. Bureau of Standards and there is no doubt as to its validity, customers seem to resent a service charge and feel that they are paying something for nothing. In order to promote customer relations, many water-works authorities consider it good practice to establish a minimum water rate, the first block of which includes the service charge and a quantity of water.

It is very difficult to establish meter rates which will provide sufficient revenue for a water works, without having experience data which indicate the water habits of the community served. It has often been noted that, when water companies change over to a meter-rate system, the consumption drops radically—sometimes 40 to 50%. This gives an idea of the water which is wasted when the system is not metered. Experienced water-works administrators are agreed that, when meter rates are set, study should be made to determine as nearly as possible the annual requirements of approximately two-thirds of the water takers and that the first block in rate structure—usually called the minimum rate—should include the service charge plus the amount of water obtained from the foregoing studies.

Lower rates are justified to large users, because it costs less to serve them. It is obvious that it costs less to serve one meter using 10,000 cu ft per day than it does 100 meters using 100 cu ft per day each. Sometimes large users force the issue and try to obtain lower rates by threatening to sink their own wells or to pump from a nearby stream. Then it is a question for the administration to determine whether it is for the best interests of the system to keep or lose the business. The general opinion is that the water should not be sold below cost, except up to the time the water works can sell its output above cost. That is, large users are profitable at low rates, up to the time their demand exceeds the supply and requires capital expenditures, which the revenue from the large users will not justify.

A municipally owned water utility should not be operated for profit. It should not yield revenue for any other services in the community. If it does yield a profit, its rates must be unfairly established,

making the customers of the utility pay some part of other people's taxes for the support of the community administration. This injustice is increased when the utility does not serve all the taxable districts within the municipal limits. In other words, the water utility is usually paid for, the bonded indebtedness is paid off, and the operating expenses are paid out of the monies received as water revenue. Therefore, the utility actually belongs to the water takers and they should reap any benefits therefrom.

There is now a bill before the House of Representatives in Massachusetts, which provides for the water utility to turn over to the city or town a certain percentage of its gross revenue, to repay the town or city for the use of its roads and property.

For the most part, private water companies are owned by groups of investors, who seek not capital gains but a security of the capital invested and continuity of a fixed income. Public sentiment would rise against them, if they tried to make exhorbitant profits.

It is the solemn duty of every water-works administrator to endeavor to earn a fair return on the fair value of the property dedicated to public use. Rates are practically the only revenue derived from the water system and must yield sufficient revenue to meet all of the yearly charges: interest on outstanding bonds, amortization payments into the sinking funds to retire bonds, operating expenses, including repairs and depreciation reserves for replacing valueless equipment, and costs of extensions and relays which are taken out of yearly revenue. In addition to these charges, private water companies must meet taxes, dividends and interest on the investment.

PROCEEDINGS

JANUARY 1947 MEETING

HOTEL STATLER, BOSTON, MASS.

Thursday, January 23, 1947

President Arthur L. Shaw in the Chair.

Secretary Knox announced the election of the following new members:

Members: William P. Brenchick, Pipe-installation Instructor and Salesman, Johns-Manville Corp., Boston, Mass.; James A. Frank, Vice-president and General Manager, National Water Main Cleaning Co., New York, N. Y.; Arthur W. Hoag, Consulting Engineer, Hoag, Stone & Associates, Burlington, Vt.; Stewart H. Newland, District Manager, Wallace & Tiernan Co., Inc., Boston, Mass.; Peter T. Peterson, Superintendent, Water Department, Maynard, Mass.

Associate: Industrias Espanolas, S.A., Manufacturers of Water Meters, San Sebastian, Spain.

Arthur C. King, Chairman of the Committee on Legislation, called attention to House Bill 24, under which 5% of the current gross receipts from water in cities, towns and districts, except the Metropolitan District, would be payable into the general municipal funds. After some discussion relative to the desirability of having the Association go on record as favoring such a bill, the President stated that he would undertake to see that some study of the matter was made.

A paper on "The Water System of the City of Philadelphia with Augmented Facilities" was read by Nathan B. Jacobs, President, Morris Knowles, Inc., Pittsburgh, Pa.

James E. Munroe, Treasurer, Munroe-Langstroth, Inc., North Attleborough, Mass., gave a talk on "The Construction of Waterworks Structures."

FEBRUARY 1947 MEETING

HOTEL STATLER, BOSTON, MASS.

Thursday, February 27, 1947

President Arthur L. Shaw in the Chair.

Secretary Knox announced the election of the following new members:

Members: Gerald W. Blakeley, District Engineer and District Manager, Johns-Manville Corp., Boston, Mass.; Albert E. Bragger, A. E. Bragger Construction Co., Greenwood, R. I.; Paul J. Cerny, Engineer, Pacific Flush-Tank Co., New York, N. Y.; Samuel M. Clarke, Consulting Engineer, Greeley & Hansen, Chicago, Ill.; Adolph Damiano, Principal Assistant Engineer, Teaneck, N. J.; George B. Garrett, Jr., Chemist, Metcalf & Eddy, Boston, Mass.; Frank W. Jones, Consulting Engineer, Havens & Emerson, Cleveland, O.; William T. Morrissey, Chairman, Metropolitan District Water Supply Commission, Boston, Mass.; Charles E. Renn, in Charge of Sanitary Laboratories, Sanitary Engineering Dept., Johns Hopkins University, Baltimore, Md.; Harold E. Willis, Foreman, Water Department, Walpole, Mass.; Seymour Squires, Secretary and Controller, Naugatuck Water Co., Naugatuck, Conn.; John F. Finn, Water Registrar, Malden, Mass.; Fred P. Stradling, Manager, Bristol County Water Co., Bristol, R. I.; Joseph J. Swiatlowski, Engineer, Water Department, Chicopee, Mass.; Elliott J. MacNeil, Water Department, Gloucester, Mass.

Associate: Liquid Conditioning Corp., Linden, N. J.

Mr. Knox stated that, with these additions, the total membership of all grades had become 861.

A talk was given by Captain R. R. Marken, District Chaplain, First Naval District, Boston, Mass., for the Red Cross Fund.

A paper on "Increase in Water Facilities to Meet Wartime Demands at Newport, R. I." was read by Harold E. Watson, Commissioner, Newport Water Works, Newport, R. I.

Hermann Behr, District Safety Director, Lumbermen's Mutual Casualty Co., Boston, Mass., gave a talk on "Plugging up Leaks."

March 1947 Meeting
Hotel Statler, Boston, Mass.
Thursday, March 20, 1947

President Arthur L. Shaw in the Chair.

Secretary Knox reported that Fred A. Darling, member, who had been Superintendent of the Franklin, Mass., Water Department for more than 40 years, passed away on March 10, 1947.

The Secretary announced the election by the Executive Committee of the following new members:

Members: Clarence L. Ahlgren, Mechanical Engineer, Water Department, Manchester, N. H.; Harold H. Bean, Superintendent, Water Works, Derry, N. H.; Frederick B. Atherton, Assistant City Engineer, Rutland, Vt.; Edmund H. Brown, Chief of Survey Party, Metropolitan District Water Supply Commission, Natick, Mass.; Roger C. Brown, President, Clarence Blair Associates, Consulting Engineers, New Haven, Conn.; Russell C. Clement, Wallace & Tiernan Co., Inc., Bloomfield, N. J.; Otis D. Fellows, Chief Engineer, State Planning Board, Boston, Mass.; Henry N. Halberg, Hydraulic Engineer, U. S. Geological Survey, Jamaica Plain, Mass.; John A. McGonnell, Superintendent, Water Department, Lynnfield, Mass.; Jack E. McKee, Consulting Engineer, Camp, Dresser & McKee, Boston, Mass.; Frederick E. Thrall, Engineer and Land Surveyor, Manchester, Conn.; John K. VanBrunt, Sales Engineer, Wallace & Tiernan Co., Inc., Boston, Mass.

Associate: Engineering Sales Corp., Boston, Mass. (Fred S. Gibbs, President).

Upon motion of Karl R. Kennison, duly seconded, it was voted that Past-Presidents Clark, Kingsbury, Carlton, Shaw and Sampson be elected to the Nominating Committee.

A paper on "Community Planning" was read by Flavel Shurtleff, Associate Professor of Administration and Legislation of Planning, Massachusetts Institute of Technology, Cambridge, Mass.

A paper on "Contractor's Problems in Construction of Water Distribution Systems" was read by Eugene M. Reppucci, President, C. Reppucci & Sons, Inc., General Contractors, Boston, Mass.

Superintendents' Round Table

During recent months several interesting and timely topics were brought before the Round Table, which aroused much thought-provoking discussion and exchange of opinion.

The subject at the November, 1946, meeting was "Nuisance Charges in Water-Works Operation", presented by Roger W. Esty of Danvers. Included in this category were turn-on and -off charges, repair charges, meter rental and testing, service-connection repairs, trip calls, and demand and summons charges.

In December, Howard C. Mandell of New Bedford spoke on "Labor Relations in Water Works", emphasizing the difficulties encountered in water-works operations with underpaid personnel.

Albert E. Casey of Stoneham led the January, 1947, meeting with "Public Relations in Water Works". In this paper Mr. Casey advised the preparing of a program covering long-range planning, together with a comprehensive plan of immediate needs, and the selling of these ideas to the town officials.

At the February meeting, John B. Kennedy of Winthrop opened up the subject of "Water Rates" and the necessity of establishing just and equitable rates, that will be fair to the water takers, the investors and the administrators of the water system.

The subject of "Unaccounted Water" was presented to the March meeting by Harry W. Dotten of Winchester. Water which gets by the meter without registering, water which is wasted or leakage which is undetected calls for continual preventive effort on the part of the superintendent. Consensus of opinion from those at the meeting indicated that to account for 80 to 85% of all water delivered was considered well done.

C. Roger Pearson